

SCHOOL OF
CIVIL ENGINEERING

INDIANA

DEPARTMENT OF TRANSPORTATION

JOINT HIGHWAY RESEARCH PROJECT

Part 2 Final Report

FHWA/INDOT/JHRP-92-25

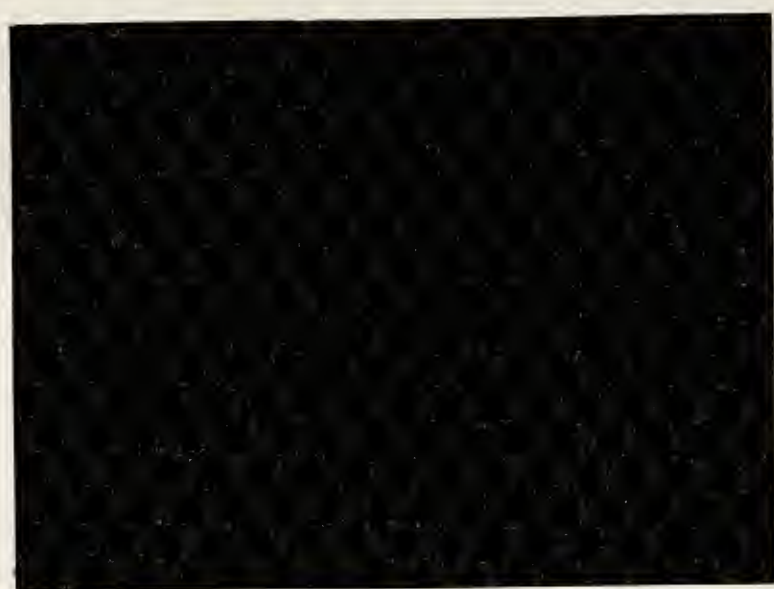
Strand Debonding in Pretensioned Beams
- Precast Prestressed Concrete Bridges
with Debonded Strands

Simply Supported Tests

O.A. Abdalla, J.A. Ramirez, and R.H. Lee



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Part 2, Simply Supported Tests

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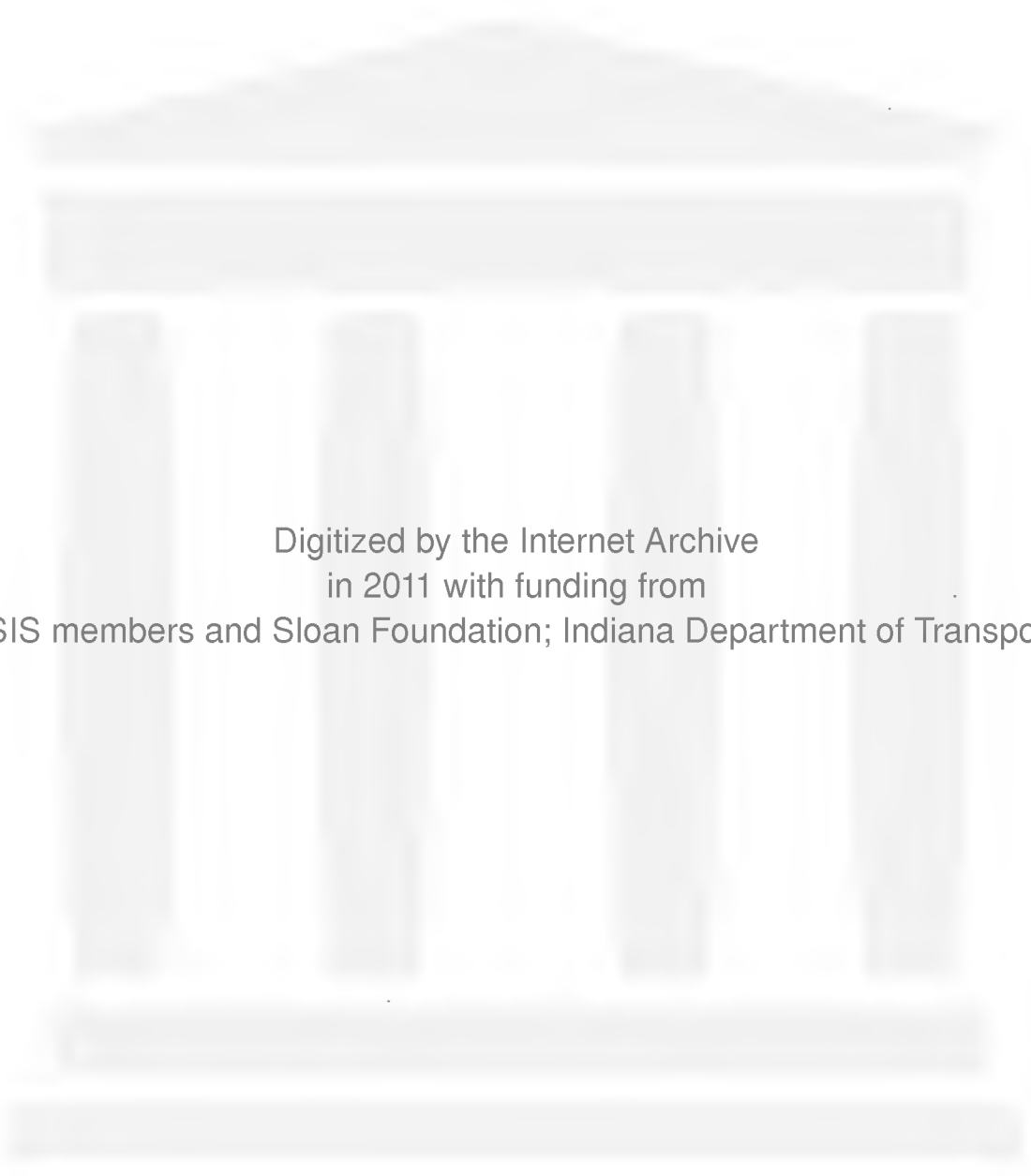
Attached is Part 2, of 2, Final Report of a research project entitled, "Strand Debonding in Pretensioned Beams" By O.A. Abdalla, J.A. Ramirez, and R.H. Lee. The report considers the comments of the advisory committee.

Respectfully submitted,

Julio A. Ramirez, and R.H. Lee, Co-Principal Investigators

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16. Abstract This report summarizes an experimental investigation regarding the effects of strand debonding on the flexure and shear behavior of simply supported precast pretensioned bridge members composite with a cast-in-place deck slab. Five specimen sets were fabricated and tested to failure as simply supported members under a single concentrated load. Four specimen sets consisted of Type-I AASHTO girders composite with a cast-in-place deck slab. One specimen set consisted of Indiana State Type CB-27 box girders also composite with a cast-in-place deck slab. Each specimen set consisted of two identical beams with different strand debonding schemes near the ends. In each set, one beam had the strands bonded throughout the entire length. The other one had some percentage of the strands debonded near the ends. The current ACI/AASHTO requirements for flexure and shear design of pretensioned bridge girders with debonded strands were examined.			
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NOTATION

A_v	= cross-sectional area of the stirrups
b_w	= web width of the girder
d	= distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement ($\geq 0.8 H$)
d_p	= nominal diameter of prestressing strand
f_d	= stress due to unfactored dead load, at extreme fiber of section where tensile stresses are caused by externally applied loads
f_{pc}	= compressive stress at the centroid of the composite section, or at the junction of the web and flange when centroid lies within the flange, due to both prestressing and the moment resisted by the precast member acting alone
f_{pe}	= compressive stress, due to prestressing, at extreme fiber of section where tensile stresses are caused by externally applied loads
f_{ps}	= stress in prestress reinforcement at nominal strength
f_{se}	= effective stress in prestressed reinforcement
f_{pu}	= specified tensile strength of prestressing strands, psi
f_{py}	= specified yield strength of prestressing strands, psi
f_y	= yield strength of nonprestressed reinforcement
f'_c	= compressive strength of concrete
H	= total depth of composite beam

- I = moment of inertia of the composite section
- l_d = development length of prestressing strand
- M_{cr} = moment causing flexural cracking at section due to applied loads
- s = stirrup spacing
- V_{ci} = nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment
- V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in
- V_d = shear force at section due to unfactored dead load
- V_i = factored shear force at section due to externally applied loads
- V_s = nominal shear strength provided by web reinforcement
- y_t = distance from the centroid of the section to extreme fiber in tension
- v = $\frac{V}{b d}$
- V = shear force at the point of bar cutoff
- b = beam width
- As_1 = area of continuous steel
- As_2 = area of steel cut off
- x = the distance between the point of bar cutoff and the end reaction
- K_1 = constant to account for nonuniformity of stress distribution over the
- M = bending moment at the point of steel cutoff
- z_1 = internal moment arm after steel cutoff

z_2 = internal moment arm before steel cutoff

x_t = distance over which redistribution of forces occurs

A_{s1} = area of main steel after curtailment

A_{s2} = area of main steel before curtailment

V'_{cr} = shear cracking resistance of the same beam with continuous steel
of area equal to that following curtailment

ABSTRACT

This report summarizes an experimental investigation regarding the effects of strand debonding on the flexure and shear behavior of simply supported precast pretensioned bridge members composite with a cast-in-place deck slab.

Five specimen sets were fabricated and tested to failure as simply supported members under a single concentrated load. Four specimen sets consisted of Type-I AASHTO girders composite with a cast-in-place deck slab. One specimen set consisted of Indiana State Type CB-27 box girders also composite with a cast-in-place deck slab. Each specimen set consisted of two identical beams with different strand debonding schemes near the ends. In each set, one beam had the strands bonded throughout the entire length. The other one had some percentage of the strands debonded near the ends.

The current ACI/AASHTO requirements for flexure and shear design of pretensioned bridge girders with debonded strands were examined.

CHAPTER 1

INTRODUCTION

This report addresses Phase 2 of the research study "Behavior of Pretensioned Bridge Members with Debonded Strands". Part 1 of the study focussed on the behavior of precast pretensioned bridge members made continuous with a cast-in-place slab and diaphragm (Abdalla et al [1992]). Four continuous specimens were fabricated and tested. Shear as well as flexural capacity of continuous pretensioned bridge members with debonded strands were studied. The combined effects of time-dependent creep and shrinkage deformations on the behavior of the beams at the continuous supports were also investigated. The current AASHTO criteria for limiting the stress at the extreme compression fiber near the continuous supports to allowable working stress values on the load carrying capacity of continuous members was also investigated.

In this report, the shear and flexural behavior of simply supported precast pretensioned girders, with debonded strands, composite with a cast-in-place slab, will be studied. An experimental program carried out to determine whether the strand debonding techniques has a detrimental effect on the load-response behavior of prestressed beams, will be amply discussed. Also the ACI/AASHTO provisions for the development length of the prestressing strands in pretensioned members with debonded strands will be examined with the aid of the results obtained in this study.

The purpose of Phase 2 of the research study was to evaluate the flexural and shear behavior of simply supported pretensioned beams with debonded strands. The behavior of debonded beams is compared to that of identical beams with fully bonded strands. The possible premature failure of the debonded beams due to lack of strand anchorage and the current ACI/AASHTO provisions for development length of prestressing strands are also investigated.

Five sets of specimens were tested to failure as simply supported members under a single concentrated load. Each specimen set consisted of two beams. One beam had all strands bonded throughout its entire length. The other had some of the strands blanketed near its end. Specimen Sets 1, 2, 3, and 5 were Type-I AASTHO girders with a 4×48 inch cast-in-place concrete slab. Specimen Set 4 consisted of Indiana State Type CB-27 box girders with a 4×36 inch cast-in-place slab.

The debonded beams in Specimen Sets 1 and 2 had 6 strands blanketed (50% debonding). In Specimen Set 3, the debonded beam had 8 strands debonded (67% debonding); and in 4 it had 10 strands (50%) debonded near its end. In Specimen Set 5, one beam had 0% debonding while the other had 6 strands (50%) debonded near each end.

In Chapter 2 of this report a literature review of relevant works is presented. The results of the experimental program are given in Chapter 3. The experimental results are thoroughly analyzed and compared with predicted values from current AASHTO specification in Chapter 4. The summary and conclusions from this phase of the research study are given in Chapter 5.

CHAPTER 2

BACKGROUND

2.1 Introduction

Strand debonding in a pretensioned prestressed beam is analogous to the bar curtailment technique often used in reinforced concrete beams. Both methods introduce the so called intermediate anchorage. These procedures induce high stress concentration, at the point of bar cutoff in reinforced concrete members or strand debonding in pretensioned beams, which may cause a deleterious effect on the ultimate strength of the beam. Evidence of reduced shear strength as well as loss of ductility when bars are cut off in the tension zone of a reinforced concrete beam has been reported in the literature. Herein, a review of the previous studies conducted on concrete beams with curtailed bars will be given first, followed by the discussion of the published work on strand debonding in pretensioned prestressed beams.

2.2 Curtailment of Reinforcing Steel

2.2.1 1959 Texas Tests

Ferguson and Matloob [1959] studied the effect of bar cutoff on the shear strength of non-prestressed reinforced concrete beams without web reinforcement. Simple span rectangular beams with several sizes and lengths and a wide variety of reinforcing steel arrangements were tested. It was reported that, when bars were cut off, flexural cracks

started at the cutoff point at a lower load than predicted. Soon thereafter this crack propagated in an inclined direction and finally resulted in failure of the beam. This premature development of the inclined crack was associated with the bar cutoff point. Beams with full length bars developed the full calculated moment capacity.

This loss in shear capacity was attributed to the additional longitudinal tension introduced in the concrete, at the cutoff point, when tensile stresses were transferred from the cutoff bars to the continuous bars. It has been found that the shear strength of the concrete beams decreases as the area of the steel cut off increases.

According to Ferguson et al [1959] the total diagonal tensile stress, v_t , at the bar cutoff point is given by:

$$v_t = [v + v \frac{x}{K_1 d} (\frac{A_{s2}}{A_{s1} + A_{s2}})] \quad (2.1)$$

where:

$$v = \frac{V}{b d}$$

V = shear force at the point of bar cutoff

b = beam width

d = beam effective depth

A_{s1} = area of continuous steel

A_{s2} = area of steel cut off

x = the distance between the point of bar cutoff and the end reaction

K_1 = constant to account for nonuniformity of stress distribution over the cross-section of the beam

The first term in Equation (2.1) is the ordinary shear stress caused by the applied loads at the point of steel cutoff. The second term gives the increase in shear stress caused by reinforcement curtailment. However, the value of the constant K_1 was not given.

2.2.2 1969 Alberta Study

An exploratory test program to study the effect of terminating part of the tensile steel on the shear strength of reinforced concrete beams had been conducted by Baron [1969]. Baron tested single span reinforced concrete beams in which an additional area of tensile steel was placed in the central portion of the beam. The additional short bars were terminated in the shear span of the simple beam. It was found that increasing the number of the additional short bars had the effect of decreasing the shear strength of the beams. It was also noticed that inclined cracks occurred in the web of the concrete beams containing the stopped bars, at the point of cutoff, before the section had cracked due to flexure. These findings confirmed the results obtained by Ferguson et al [1959]. It was concluded that diagonal tension failure was prone to occur at the locations of reinforcement termination. The sudden transfer of tensile stresses at the point of bar cutoff aggravated the shear stress at that location. An additional tensile force due to the shift in the neutral axis existed at the point of cutoff. At the point of steel cutoff the internal moment arm would be less on the side having the larger amount of steel. Therefore, to develop the same resisting moment at that section the tensile force in the longitudinal steel before curtailment will be greater than that after curtailment. Baron related the sudden increase in shear stress at the point of steel cutoff to the reduction in

the steel tensile force at the cutoff point.

According to Baron the average shear stress in the beam web due to the decrease in the steel tensile force is given by:

$$v_t = \frac{M}{b x_t} \frac{z_1 - z_2}{z_1 z_2} \quad (2.2)$$

The total shear stress acting at the point of bar cutoff is given by:

$$v_t = \frac{V}{b d} + \frac{M}{b x_t} \frac{z_1 - z_2}{z_1 z_2} \quad (2.3)$$

where:

V = shear force acting at the point of steel cutoff

M = bending moment at the point of steel cutoff

b = beam width

d = beam effective depth

z_1 = internal moment arm after steel cutoff

z_2 = internal moment arm before steel cutoff

x_t = distance over which redistribution of forces occurs

2.2.3 1972 Imperial College Approach

Baron's approach was followed later by Reagan and Mitra [1972] for the evaluation of the shear capacity of reinforced concrete beams with curtailed longitudinal steel. The shear cracking resistance of a beam with curtailed steel was given by:

$$V_{cr} = V'_{cr} \left[1 - 0.2 \frac{M}{V d} \left(\sqrt{\frac{A_{s2}}{b d}} - \sqrt{\frac{A_{s1}}{b d}} \right) \right] \quad (2.4)$$

V_{cr} = shear cracking resistance of a beam with curtailed steel

V'_{cr} = shear cracking resistance of the same beam with continuous steel of area equal to that following curtailment

A_{s1} = area of main steel after curtailment

A_{s2} = area of main steel before curtailment

M = bending moment at the point of cutoff

V = shear force at the point of cutoff

b = beam width

d = beam effective depth

It was shown that Equation (2.4) was in excellent agreement with the test results reported by Baron [1969]. Reagan and Mitra suggested that the distance over which redistribution of forces occurs, x_t in Equations (2.2) and (2.3), is to be taken as equal to the effective depth of the beam.

This study indicated that, the effect of curtailment of main steel on shear cracking resistance is greater than that which would be predicted by the 1969 British Standards.

2.3 Strand Debonding

It has been stated earlier that strand debonding in prestressed beams is similar to bar cutoff in non-prestressed reinforced concrete beams. It is expected that the adverse effect of bar curtailment in non-prestressed beams will be also associated with strand debonding in prestressed beams. Thus far, the effect of strand debonding on the shear strength of pretensioned precast beams has not been given enough attention in the technical literature. Very little experimental work has been carried out to evaluate the

shear strength of prestressed beams with blanketed strands.

2.3.1 1965 PCA Tests

Kaar and Magura [1965], as part of a wider study investigated the effect of strand debonding on the shear strength of pretensioned concrete girders. The investigation involved five half-scale Type III AASHTO bridge girders, 34 ft. long with a 39×3 inch composite concrete slab. Three girders were designed and tested for the study of flexural behavior of prestressed beams with debonded strands. The prestressing steel used was seven-wire, stress relieved, Grade 270 ksi strand of $3/8$ in. diameter. All three beams were over-reinforced with stirrups to prevent the interference of shear distress with the flexural behavior. These beams were tested statically to failure after 5 million cycles of the design service load over a single span, 33 ft. long. It was found that debonding did not affect the behavior of the girders in the service load range. After cracking, however, the overall stiffness of the girder was considerably reduced. Wider flexural cracks were observed in girders with blanketed strands. It was concluded that the requirement for embedment length of strands as specified by the ACI 318-63 code was inadequate when applied to blanketed strands. The flexural behavior of girders with embedment lengths twice those required by the ACI 318-63 code, was found to be similar to the behavior of girders with fully bonded strands. The development length of the prestressing strand according to the ACI 318-63 code was 5.5 ft. The shear investigation involved static testing to failure of two girders similar to those tested for the flexure study, except that the amount of web reinforcement was reduced to determine the effect of blanketing on the shear capacity. The stirrups in these girders

had a spacing $1\frac{1}{2}$ times that required by the 1963 ACI Building Code. The results from these tests indicated no detrimental effects of blanketing upon the shear carrying capacity of the pretensioned girders. It is worth mentioning that the blanketed strands in the debonded girder had embedment lengths twice those required by the ACI 318-63 Code.

2.3.2 1971 Glasgow Tests

The shear strength of uniformly prestressed pretensioned concrete I-beams with debonded strands was investigated by Krishnamurthy in 1971. All beams were simply supported on a span of 9 ft. and subjected to a two-point loading with a shear span of 20 inches. Diagonal tension failure occurred in all the specimens tested in this study. It was found that increasing the number of debonded tendons in the bottom flange, decreased the bearing capacity in shear of the uniformly prestressed beams. On the other hand, increasing the number of debonded strands in the top flange increased the shear strength of the girders when compared to a similar girder with fully bonded strands. These tests were conducted on beams without shear reinforcement.

2.3.3 1975 Tulane Strand Blanketing Report

Dane and Bruce [1975] performed tests on nine composite concrete girders to study the effectiveness of debonding the prestressing strands as an alternative to draped strands in pretensioned prestressed concrete beams. Six were half-scale Type III AASHTO girders, 34 ft. long and simply supported on 33 ft. span. The remaining three were Type II AASHTO girders, 50 ft. long and simply supported on 48 ft. span. Two of the first six girders had draped strands, whereas the other four had straight strands

which were blanketed near the ends of the girders. One of the full scale Type II AASHTO girders had draped strands and the other two had blanketed strands. The stirrups in these specimens were designed to ensure a flexural mode of failure. All of the girders were loaded with two equal concentrated loads applied at the third points of their clear spans. The blanketed strands in two of the test specimens were surrounded with a cage of mild reinforcing steel at the point where blanketing ended. In the other two the blanketed strands were anchored with internal locking devices at the debonding point. In all the debonded beams, one development length as specified by the ACI 318-63 code was provided for the blanketed strands. It was found that either draping or blanketing did not significantly affect neither flexural cracking nor inclined shear cracking. It was also noticed that no significant reduction in flexural strength occurred when using blanketed strands in place of draped strands. This was related to the beneficial effects from wrapping the strands and the internal anchorage devices. Dane and Bruce concluded that debonding the prestressing strands was an effective method that could be used in lieu of draping in pretensioned concrete beams. However, the effect of debonding the strands on the shear strength of pretensioned girders was not addressed in this investigation.

2.3.4 1979 PCA Tests

Rabbat et al [1979] tested six full-size Type II AASHTO girders with a composite concrete slab at the Construction Technology Laboratories of the Portland Cement Association. Two of the test girders had draped strands. In the other four, only straight strands blanketed at the ends of the girders were employed. The shear reinforcement in

all beams consisted of #4 U-stirrup spaced at 6 inches on center. The behavior and strength of the test girders under the effect of cyclic loading was investigated in this study. The specimens were simply supported and loaded with four concentrated loads, placed symmetrically about the center line of the span. The test girders were subjected to 5 million cycles of the full service load. Subsequently, the specimens were tested statically to destruction. It was found that, pretensioned beams with debonded strands performed satisfactorily if adequate strands development was provided. When no tension was allowed in the precompressed tensile zone, one development length as suggested by the ACI 318-77 specifications was found to be adequate. The fatigue life of the beam was reduced when tensile stresses were allowed in the bottom fibers. It was recommended that the development length specified by ACI 318-77 should be doubled in such beams. The shear strength of pretensioned concrete beams with debonded strands was not directly investigated in this study as the specimens were designed to ensure flexural failure.

2.3.5 1983 Auckland Shear Behavior Tests

An experimental investigation to study the behavior of pretensioned I-beams with debonded strands was carried out by Dale in 1983 at the University of Auckland, New Zealand. It was reported that debonding the prestressing strands generally reduced the ultimate capacity of the test beams in shear. Large strains and wide inclined cracks were observed in beams with debonded strands. The addition of extra stirrups over the transfer length of the debonded strands markedly improved the performance of the prestressed concrete beams. The diagonal crack widths were smaller and stirrup and

compression zone strains were less severe.

2.3.6 1987 Purdue Fatigue Study

The fatigue behavior of six prestressed box beams with various embedment lengths was investigated by Pensinger in 1987 at Purdue University. Different development lengths were used for the debonded strands. Pensinger investigated the validity of the ACI 318-89 Specifications requiring an embedment length for debonded strands twice that required for fully bonded strands when tensile stresses exist in the precompressed zones under service load conditions.

The test girders were loaded cyclically up to 5 million cycles with a nominal tensile stress of $6 \sqrt{f'_c}$ in the precompressed zone. The stress level was increased to $8 \sqrt{f'_c}$ when failure did not occur in the specimen. The specimen was then subjected to cyclic loading until significant fatigue damage was observed. It was concluded that the behavior of beams with $1.5 l_d$ development length was similar to that of beams with $2 l_d$ development length. l_d is the development length required by the ACI 318-89 Code for fully bonded strands. The author suggested that $1.5 l_d$ of embedment length should be used for debonded strands in members where $2 l_d$ is required by the ACI 318-89 Building Code. The shear behavior was not addressed in this study.

2.4 Development Length

In pretensioned members, prestressing is achieved by transferring the prestressing force imparted by the strands to the member by virtue of the bond between the concrete and the prestressing steel. The distance, from the end of the member, over which the effective prestressing force is developed is referred to as the transfer length. An

additional bond length is always required in order that the ultimate strength of the strand be developed at the critical section for flexure. This additional length is called flexural bond length. The sum of these two lengths, the transfer length and the flexural bond length, is the development length of the strand.

Accurate estimation of transfer and development lengths of prestressing strand is essential for the prediction of the shear and flexural strength of pretensioned beams. Insufficient development length could cause excessive strand slippage and alter the overall behavior and mode of failure.

Several studies have been conducted during the last three decades trying to determine the adequate embedment length of the prestressing strands. A brief description of these studies will be given in the following sections.

2.4.1 Hanson and Kaar [1959]

Hanson and Kaar carried out an experimental investigation of flexural bond in beams pretensioned with seven-wire strand of 1/4, 3/8, and 1/5 inch diameter. The test program involved 47 pretensioned beams tested under static loading to evaluate the effect of strand diameter and embedment length on bond performance. The concrete strength of the specimens ranged from 3700 psi to 7800 psi. It was found that strand size and embedment length have a considerable influence on the value of the average bond stresses at which general bond slip occurs. The test results showed that beams with adequate embedment length failed in flexure by crushing of the concrete after yielding of the steel before general bond slip occurred. As the embedment length decreased, failure occurred at lower loads due to slippage of the strands. Hanson and

Kaar recommended that the following embedment lengths be provided from the location of the applied load to beam end if the ultimate strength of strand was to be developed by beam flexure before general bond slip occurred: 70 in. for 1/4 in. strand ($f_{py} = 275$ ksi); 106 in. for 3/8 in. strand ($f_{py} = 263$ ksi); and 134 in. for 1/2 in. strand ($f_{py} = 263$ ksi). However, Martin and Scott [1976] after reevaluating the test data reported by Hanson and Kaar stated that the minimum embedment length for 1/2 in. (270 ksi) strand should be 151 inches. It must be pointed out that the beams of high strength tested by Hanson and Kaar failed by general bond slip, whereas the low strength concrete beams failed in flexure by crushing of the concrete after yielding of the steel. The prestressing steel stress immediately after transfer, which was gradual in these tests, did not reach 70 percent of the ultimate tensile strength of the strand. These effects contributed to the satisfactory performance of the specimens.

2.4.2 ACI/AASHTO [1989] Provisions

The ACI/AASHTO specifications for development length are based primarily on the work of Hanson and Kaar [1959]. The ACI/AASHTO provisions require that the bonded embedment length of the prestressing strands from the critical section under consideration shall not be less than:

$$l_d = \frac{f_{se}}{3} d_p + (f_{ps} - f_{se}) d_p \quad (2.5)$$

where:

l_d = development length of the prestressing strand (inch)

f_{ps} = stress in prestress reinforcement at nominal strength (ksi)

f_{se} = effective stress in prestressed reinforcement (ksi)

d_p = nominal diameter of prestressing strand (inch)

Equation (2.5) was based on tests of beams with all strands fully bonded from the section of maximum bending moment to the beam ends. Based on the paper published by Kaar and Magura [1965] the ACI/AASHTO provisions were revised for blanketed strands. The provisions required that, where bonding of a strand does not extend to the end of the members, and design includes tension at service load in precompressed tensile zones, development length specified in Equation (2.5) shall be doubled. Based on a subsequent study by Rabbat et al [1979] it was established that doubling the development length was not necessary if tension was not allowed at service loads in the precompressed zone of the specimens.

2.4.3 Zia and Mostafa [1977]

Zia and Mostafa reanalyzed the test results from Hanson and Kaar pertaining to development length. Zia and Mostafa stated that the actual embedment length for the strands which developed the ultimate strength before a general bond slip were considerably shorter than indicated by Hanson and Kaar tests. The authors proposed the following formula for development length:

$$l_d = 1.5 \frac{f_{si}}{f_{ci'}} d_p - 4.6 + 1.25 (f_{su} - f_{se}) d_p \quad (2.6)$$

Equation (2.6) is applicable for concrete strength ranging from 2000 to 8000 psi and accounts for effects of strand size, initial prestress and concrete strength at transfer.

2.4.4 EL Shahawy and Batchelor

Current research on development of prestressing strands is being conducted at the

Florida Department of Transportation Laboratories by EL Shahawy and Batchelor [1991]. Pretensioned prestressed concrete beams were tested under static loading in flexure and shear to determine the adequate development length of the strands. Preliminary tests were conducted on fully bonded pretensioned beams reinforced with 0.5 inch diameter, low-lax, Grade 270 ksi strand with an effective prestress of 162 ksi. These tests indicated that the current development length of such strand should be 1.69 times the development length required by ACI/AASHTO [1989] provisions.

The adequacy of the present provisions of the ACI/AASHTO Codes with respect to development is controversial. In this report the results obtained from testing ten pretensioned composite girders, five of which had blanketed strands, will be used to check the adequacy of the development lengths recommended by ACI/AASHTO Codes and the recently proposed FDOT specification.

2.5 Summary

This chapter contains a brief review of the studies that have dealt with the effects of strand debonding and bar cutoff in concrete girders. It was found that, flexural cracks developed at cutoff points at lower loads than expected. Inclined cracks were observed soon thereafter and caused the early failure of the concrete beams. Beams with continuous longitudinal steel failed at the predicted ultimate load. The analytical studies conducted to evaluate the effect of bar cutoff on the load carrying capacity of reinforced concrete beams were also presented in this chapter. It was determined that, the reduction of the shear cracking capacity in beams with curtailed reinforcement was due to the sudden increase in the shear stress at cutoff points.

Previous studies on strand debonding revealed that strand debonding in pretensioned beams reduced the cracking capacity. Upon cracking, the overall stiffness of beams with debonded strands was observed to be considerably reduced. Wider cracks and higher strains were observed in beams with debonded strands. However, the addition of extra stirrups over the transfer length of debonded strands was found to improve the performance of pretensioned beams. Crack widths were smaller and stirrup and compression zone strains were less severe.

A brief summary concerning the research studies that led to the development of the ACI/AASHTO equations for embedment length of prestressing strand in pretensioned members was also presented. However, recent studies revealed that the embedment length required by the current ACI/AASHTO Codes for prestressing strands is not adequate. It is proposed that the development length of fully bonded strands should be seventy per cent in excess to the requirement of the ACI/AASHTO Codes.

The next chapter is an outline of the experimental program carried out to evaluate the behavior of pretensioned bridges with debonded strands tested as simply supported members. Particular emphasis will be given to the effects of strand debonding on the shear strength of these members.

In Chapter 4, a comparison of the experimental results of the tests with the theoretical analysis based on the ACI/AASHTO Codes provisions will be presented. Both the flexural and shear capacity of the simply supported pretensioned girders will be evaluated and discussed.

Chapter 5 contains a summary of this phase of the study, the conclusions drawn from the test data, and future research needs pertaining to the use of strand debonding in simply supported pretensioned bridge members.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 Introduction

In phase 1 of this research study (Abdalla et al [1992]), the testing of four of the specimen sets as continuous members (Specimen Sets 1,2,3, and 4) was decided. After each continuous test, each beam of the corresponding specimen set was further tested over a simply supported span of 17.5 ft. In Specimen Set 5 each beam was tested only as simply supported over a span of 24 ft. The simple supports were consisted of greased rollers placed between two steel plates as described in Figures 3.1 and 3.2. The specimen details are shown in Figures 3.3-3.5. Each beam was tested using a 600 kip Baldwin testing machine. The composite girders of Specimens sets 1, 2, 3, and 4 were positioned in such a way that the applied load would cause the critical region for high shear force to occur at the end that had not been damaged by the continuous tests. The girders with blanketed strands were tested so that the end with blanketed strands would be in the region where shear failure was likely to occur.

3.2 Materials

3.2.1 Concrete

The girders were cast using the standard 6000 psi concrete mix for pretensioned bridge members in the State of Indiana. The concrete compressive strength was

monitored using the standard 6×12 test cylinders. The compressive strength of the concrete used in the precast beams and the cast-in-place slab and diaphragm, for the different test specimens, is shown in Figures 3.6-3.10.

3.2.2 Prestressing Steel

The prestressing steel, in both girders, consisted of stress relieved (Specimen Set 1) and Lo-Lax (Specimen Sets 2, 3 and 4) Grade 270, uncoated seven-wire strands, 0.5 inch diameter (cross-sectional area of 0.153 in²). The stress-strain behavior of the strand is shown in Figures 3.11-3.13. Strains were measured by means of electrical resistance strain gages attached to the strand as in the test beams. The yield stress, the ultimate strength, and the modulus of elasticity of the strand were determined from these tests.

3.2.3 Non-Prestressed Reinforcement

Standard deformed Grade 60, #6 bars were used as the nonprestressed top reinforcement in the precast beams and the deck slab reinforcing mat. The properties of these bars were determined from tension tests. The stress-strain behavior of the #6 bars is shown in Figures 3.14-3.17.

The stirrup reinforcement consisted of deformed Grade 60, #3 bars. The stress-strain curve for the stirrup steel is shown in Figures 3.18-3.20.

3.3 Simply Supported Tests

The experimental setup for the simply supported tests is shown diagrammatically in Figures 3.1 and 3.2.

In these tests, the load was applied monotonically in small increments until the specimen could not sustain any additional load. The applied load was measured by a load cell with a 300 kip capacity.

Deflections under the point load and at debonding points location were measured using LVDT's at both sides of the beam. The location of these measurement devices is shown in Figures 3.21-3.23.

Strains in the prestressing strands, stirrups, and deck slab longitudinal steel were monitored using electrical resistance strain gages. Concrete strains at the top fiber of the cast-in-place slab were measured at the point load and at debonding points location as shown in Figures 3.21-3.23.

The prestressing strand slip was measured using mechanical deflection gages mounted on the strands protruding from the end of the beam. Figures 3.24-3.28 illustrate the position of these gages.

During each test, the applied load was increased in small increments. At each increment, the external load, strains, and deflections were recorded using an automated data acquisition system along with manual reading of the prestressing strand slip data. New and extended cracks were identified and marked on the surface of the beam together with the corresponding load level. The performance of the test girders will be discussed in the following sections.

3.3.1 Specimen Set 1

In Specimen Set 1, one beam had all 12 strands fully bonded. The other had 6 of the strands debonded near the end EA (50% debonding) as shown in Figure 3.29. In this specimen, transverse reinforcement was not provided in the cast-in-place slab. In both beams longitudinal cracks developed in the slab at its junction with the flange of the precast I-beam. Further loading caused failure of the slab to occur prior to the actual girder failure. Slab failure occurred at a load of 185 kips in the 0% debonded beam, and at a load of 135 kips in the 50% debonded beam. Data was recorded for both beams up to the failure load of the slab.

3.3.1.1 Cracking

Web-shear cracking of the fully bonded girder occurred at an applied load of 150 kips at a distance of 22 inches from support. Flexure-shear cracking was observed at a load of 165 kips at a distance of 73 inches from the support. Cracking patterns of the fully bonded beam, before and after slab failure, are shown in Figures 3.30-3.31. In the beam with 50% debonding, web-shear cracking was observed at a load of 120 kips at a distance of 22 inches from the support. The first flexure-shear crack occurred at a load of 113 kips near the debonding point, a distance of 62.5 inches from the support. Crack patterns of the 50% debonded beam, before and after slab failure, are shown in Figures 3.32 and (3.33).

3.3.1.2 Deflections

The load-deflection relationship for the 50% debonded beam and the fully bonded beam is shown in Figures 3.34-3.37. In both beams the measured deflection was

proportional to the applied load until web-shear and flexure-shear cracking occurred. Large increase in deflection was observed when cracking occurred. Comparison of the deflection of the two beams shows that, up to the initiation of the first crack the two beams had the same deflection. The second stage of load-deflection curves shows that at the same applied load the 50% debonded beam had almost twice the deflection as the fully bonded beam.

3.3.1.3 Concrete Top Fiber Strains

Concrete strains were monitored by surface strain gages in the compression region ,at the top fiber of the cast-in-place slab, at the point load and at debonding points location as shown in Figure 3.21. Figures 3.38-3.41 show the concrete strain versus the applied load for both tests. These gages show that similar behavior was exhibited by the strains in the concrete for the different beams. Substantial increase in strain occurred when flexure-shear cracking opened.

3.3.1.4 Stirrup Strains

The instrumented stirrups near the end of the test girders are shown in Figure 3.42. Load versus stirrup strains curves are given in Figures 3.43 and 3.44 for the fully bonded girder and the 50% debonded girder, respectively. Comparison of the stirrup strains at beam failure shows that higher strains occurred in the stirrups of the 0% debonded beam. None of the stirrups yielded in these tests.

3.3.1.5 Longitudinal Bar Strains

Three deck bars were instrumented near the applied load to measure the longitudinal bar strains as illustrated in Figure 3.45. Typical curves are shown in

Figures 3.46 and (3.47) for both the fully bonded girder and the 50% debonded girder before failure of the cast-in-place slab. The strain in the slab longitudinal steel, in the fully bonded beam, varied linearly with the applied load up to the slab failure. In the debonded beam linear behavior occurred only up to flexure-shear cracking.

3.3.1.6 Strand Strains

The load-strain relationships for the prestressing strands are presented in Figures 3.48-3.53. Before flexure cracking occurred the change in the strand strain was small and proportional to the applied load. Sudden increase in the strand strain occurred when flexure cracking developed in the bottom flange of the beams.

3.3.1.7 Strand Movement

Four of the strands in each beam were monitored during each test to detect their movement as shown in Figure 3.24. In the 0% debonded beam test, no movement of the strands was recorded until failure of the girder had occurred. In the 50% debonded beam, strand movement occurred when the first crack appeared, which was a flexure-shear crack at an applied load of 113 kips. No movement was detected for the two strands which were fully bonded throughout the length of the beam. Movement was recorded for the two debonded strands, and Figure 3.54 shows the load versus strand movement curves for the two debonded strands. The movement of the debonded strands was the sum of two quantities: slippage of strands in the bonded region and the movement of the strands inside the plastic tubes when crack opened in the debonded region or at the debonding points. In this study the separate value of the two effects was not determined.

3.3.1.8 Failure Loads

Both girders were tested to failure even though failure of the slab occurred before girder failure. The 0% debonded beam failed at an applied load of 225.4 kips. Failure of the beam occurred by crushing of the top concrete fibers, which led to an explosive failure. The failure removed a section of the concrete from the bottom flange of the beam.

Failure of the 50% debonded beam occurred at an applied load of 148.2 kips. Failure of this beam was caused by flexure-shear cracks which opened near the two debonding points. Four of the fully bonded strands ruptured in the failure region of this beam.

3.3.2 Specimen Set 2

The debonded girder, in Specimen Set 2, had 4 of the strands blanketed at a distance of 6 ft. from the centerline of the support. Two additional strands were debonded at 3.5 ft. from the centerline of the support. Figure 3.55 shows the strand debonding scheme and instrumentation for the specimen.

3.3.2.1 Cracking

The first web-shear crack developed in the fully bonded girder at the 130 kip load level, 30 inches from the support. The first flexure-shear crack developed at 180 kips under the point load. At 194 kips the girder developed what became the failure crack in the form of a flexure-shear crack as shown in Figure 3.56.

The 50% debonded girder developed its first web-shear and flexure-shear cracks at lower loads than the fully bonded girder, as expected. The first web-shear crack in the debonded girder occurred at a load level of 114 kips and was located at 14 inches from the end support. The first flexure-shear crack in the debonded girder formed at the 120 kip load level. The flexure-shear crack originated at the bottom flange near the first debonding point (42 inches from the support, see Figs. 3.55 and 3.57) and then propagated diagonally toward the point load until it stopped at the web-top flange junction one foot in front of the point load. Additional loading induced cracking in the form of web-shear and flexure-shear cracks until the girder lost its capability to sustain additional loading at the 172 kip load level as shown in Figure 3.57.

3.3.2.2 Deflections

The fully bonded girder had a linear load-deflection relationship up to the load at which web-shear cracking occurred. Subsequent loading caused larger deflection with smaller load increments. Figures 3.58 and (3.59) illustrate the load-deflection relationships for the fully bonded girder.

The load-deflection relationship for the debonded beam is linear up to 120 kips, which is the load at which the girder experienced its first flexure-shear crack. After the 120 kip load, larger deflection was observed with smaller load increments as shown in Figures 3.60-3.61. At failure the debonded girder deflected almost twice that of the fully bonded girder.

3.3.2.3 Concrete Top Fiber Strains

Concrete strains were measured at the top fiber of the deck slab at the point load and at 42 inches from the support as shown in Figure 3.21. The distance 42 inches from the support corresponds to the location of the first debonding point in the 50% debonded beam. The load-strain relationships are illustrated in Figures 3.62-3.65. At the location of the debonding point, the top fiber strains for the two girders displayed little difference. At the point load the debonded girder displayed much more strain than the fully bonded girder. None of the strains reached the 0.003 limiting concrete compressive strain.

3.3.2.4 Stirrup Strains

The location of the instrumented stirrups for Specimen Set 2 is shown in Figure 3.42. Figures 3.66 and 3.67 show the stirrup load-strain behavior for the two simply

supported girders. Overall, the debonded girder displayed more stirrup strain for a given load than the fully bonded girder except for the second instrumented stirrup (EA2 and ED2). The second stirrup in the fully bonded girder displayed more strain. This was caused by a diagonal crack that crossed this stirrup, the debonded girder did not have any significant cracks that cross the corresponding stirrup. High strains were induced in the stirrups only when diagonal cracking crossed them.

3.3.2.5 Longitudinal Bar Strains

The longitudinal slab bar strains are shown in Figures 3.68 and 3.69. The fully bonded beam showed linear behavior up to the failure load. The 50% debonded beam showed linear behavior up to the formation of the flexure-shear crack. After flexure-shear cracking a reduction in strain was shown by these gages.

3.3.2.6 Strand Strains

The measured strain versus load behavior in the strands of Specimen Set 2 are shown in Figures 3.70-3.74. No significant increase in the strain was recorded before flexure cracking had occurred. In both beams large increase in the strain was observed at flexure cracking. None of the strands in the fully bonded girder reached yielding prior to the peak load. After the peak load was reached the strain gages began to malfunction due to large amount of strand slip. Strand EA5 in the beam with 50% debonding, shown in Figure 3.55, was stressed beyond the yield stress. This high strain was caused by the failure flexure-shear crack that opened near the location of the gage. It must be noted that this strand was fully bonded throughout the entire length of the beam. The strain gages began to disfunction shortly thereafter.

3.3.2.7 Strand Movement

Figure 3.25 illustrates the placement of the five dial gages used to monitor the strand slip. In the debonded girder, all of the monitored strands began to move at 120 kips, the load that corresponds to the first flexure-shear crack. When the load reached 160 kips all of the dial gages had to be removed because the strand movement was larger than their ranges. The reading of the dial gages mounted on the debonded strands included the strand movement inside the plastic tubes which occurred when cracks opened at the debonding point. Three of the monitored strands in the 0% debonded beam slipped at a load of 170 kips. In both beams, when the ultimate load was reached, the load dropped. Subsequent loading caused all of the monitored strands to slip beyond the range of the dial gages. All of the gages in both tests had to be removed. Figures 3.75 and 3.76 show the load-movement behavior of the strands.

3.3.2.8 Failure Loads

The fully bonded girder failed at a load of 194 kips due to inadequate anchorage of the required force in the strands. After the 194 kip-load was reached, the load dropped to 170 kips and subsequent reloading caused several strands to rupture due to large deformations.

The 50% debonded beam failed at a load of 172 kips. Examination of the test results indicated that failure was caused by a similar mechanism as the one observed in the specimen with 0% debonding but at a lower load level with far larger deflections.

3.3.3 Specimen Set 3

The first beam in Specimen Set 3 had all 12 strands fully bonded, while the second had 8 strands debonded (67%) as shown in Figure 3.77. The load was applied in small increments. Data was collected after each load increment was applied, at which time cracks were observed, marked, and photographed.

3.3.3.1 Cracking

Web-shear cracking in the fully bonded beam was observed at a load of 152 kips. This crack originated at a distance of 20 inches from the centerline of the end support (ED) and penetrated through the bottom flange. The applied load then dropped to 141 kips. The prestressing strands were observed to be pulling into the girder at the end of the beam immediately after the web-shear crack formed. Mechanical deflection gages were placed at the protruding ends of the strands to measure the resulting slippage as shown in Figure 3.26. Additional cracks developed in the web and also penetrated through the bottom flange when the load was subsequently increased to 152 kips. When the load reached 158 kips a diagonal crack opened at a distance of 42 inches from the support in the form of a flexure-shear crack. This crack then changed its direction and propagated along the prestressing strands a distance of 20 inches towards the end of the beam. The beam failed at 160 kips, when another inclined crack opened in the web and joined this crack in the bottom flange. The crack pattern of the beam at failure is shown in Figure 3.78.

The beam with 67% debonding was tested on simple supports, the load was applied monotonically in small steps until the beam failed. When the load reached 102 kips a

flexure-shear crack appeared at the second debonding point. At a load of 110 kips three parallel diagonal cracks opened in the web of the precast beam, between the support and the second debonding point. As soon as the load reached 118 kips three parallel inclined cracks opened in the web. One of these cracks extended to the bottom of the flange at the first debonding point. The load then dropped to 96 kips. When the load increased to 98 kips two cracks developed near the first debonding point and caused the ultimate failure of the beam. The crack pattern of the beam at failure is shown in Figure 3.79.

3.3.3.2 Deflections

The vertical deflection in both beams of Specimen Set 3 was measured at the point of application of the external load and at the first debonding point location in the beam with 67% debonding, a distance of 42 inches from the center line of the end support. The deflection was measured with LVDT's placed at both sides of the beam as shown in Figure 3.21. The load-deflection relationships for the fully bonded beam are shown in Figures 3.80 and 3.81. The deflection of the beam varies linearly with the applied load until web-shear cracking occurred. Once again, flexure-shear cracking clearly marked the limit of the load carrying capacity of both beams in Specimen Set 3.

The load-deflection curves at the location of the external load and at the second debonding point, for the beam with 67%, are presented in Figures 3.82 and 3.83 respectively. The response of this beam is similar to the beam with fully bonded strands. Prior to cracking, the response is linear followed by nonlinear response after cracking. However, comparison of the deflection curves for the two beams reveals that,

the beam with 67% debonding had larger deflections.

3.3.3.3 Concrete Top Fiber Strains

The concrete compressive strains were monitored by surface gages installed at the top of the slab. Figure 3.21 shows the location of the slab surface gages. In the fully bonded beam substantial increase in concrete strains was exhibited by these gages after the formation of web-shear cracking as indicated in Figures 3.84 and 3.85.

In the beam with 67% debonding the concrete strains in the deck slab at the external point load showed linear variation with the load up to the formation of the flexure-shear crack, at a load of 102 kips. However, the strains at the first debonding point, 3.5 ft. from the center line of the support EA, continued to vary linearly with the load until the web-shear crack formed near the end support when the load was 110 kips as shown in Figures 3.86 and 3.87.

3.3.3.4 Stirrup Strains

The strain recorded by the gages mounted on four stirrups located at the ends of the girders (see Figure 3.88), are plotted in Figures 3.89 and 3.90. As expected, these gages did not register significant strains until they were crossed by diagonal cracks. It can be noticed that the increase in stirrup strains was large and sudden. In the fully bonded beam stirrup ED3 and ED4 reached their yield strain. In the beam with 67% debonding stirrup EA4 reached the yield strain.

3.3.3.5 Longitudinal Bar Strains

The strain in the mild reinforcing steel in the slab was measured at , 6 ft. from the center line of the end support as shown in Figure 3.45. Figures 3.91 and 3.92 show the

measured strains in three different bars. For both beams the response was linear before the opening of web-shear cracking and nonlinear after web-shear cracking occurred owing to the drop in load carrying capacity.

3.3.3.6 Strand Strains

Figures 3.93-3.98 show the variation of strand strain with the applied load. In the beam with 0% debonding the strand strains vary linearly with the applied load. At the point load location sudden increase in the strand strain was noticed when a flexure crack opened in the bottom flange. Similar behavior was exhibited by the beam with 67% debonding. However, a reduction in strains due to strand movement was observed in the beam with 0% debonding as shown in Figure 3.93 and in the beam with 67% debonding as shown in Figures 3.97 and 3.98.

3.3.3.7 Strand Movement

As mentioned earlier the prestressing strands slipped into the girder as soon as web-shear cracking occurred in the 0% debonded beam. In the 67% debonded beam movement of the strands was noticed when a flexure-shear crack opened at the second debonding point at a load of 102 kips. The load-movement relationship for the instrumented strands is shown in Figures 3.99 and 3.100. It can be seen that all the instrumented strands showed significant movement when diagonal cracking occurred as it penetrated into the strand level.

3.3.3.8 Failure Loads

Based on the previous discussion, it can be concluded that cracking in the bottom flange near the ends of the girders disturbed the development of the prestressing steel

and caused bond and anchorage failure. This led to the opening of wide diagonal cracks which resulted in stressing the stirrups to their yielding points, thus causing premature shear failure due to inadequate load carrying capacity of the bottom tension reinforcement in the beams. The 0% debonded beam failed at a load of 160 kips. The beam with 67% debonding failed at a load of 118 kips.

3.3.4 Specimen Set 4

Specimen Set 4 consisted of two precast Indiana State Type CB-27 box girders with a 4×36 inch composite slab. One girder had 10 strands (50%) debonded near its end. The other had all strands fully bonded (0% debonding). The strand debonding scheme and instrumentation is shown in Figure 3.101.

3.3.4.1 Cracking

The first appearance of diagonal cracking in the beam with 0% debonding was in the form of web-shear cracking at a load of 160 kips at a distance of 16 inches from the end support. No web-shear crack was observed on the opposite side of the beam until the load reached 245 kips. At that time, two parallel diagonal cracks opened in the shear span as shown in Figure 3.102. A flexure-shear crack was also observed at a distance of 33 inches from the support. The load then dropped to 190 kips. The beam lost its ability to carry more load. At this time the test was concluded. No flexure cracks were observed under the point load.

In the 50% debonded beam the first appearance of web-shear cracking was at a load of 176 kips at a distance of 12 inches from the support. At the same load level a flexure-shear crack opened at the first debonding point and propagated along the junction of the slab and the precast beam until it reached the point of application of the external load. At this point the beam could carry no additional load. The crack pattern at peak load is shown in Figures 3.102.

3.3.4.2 Deflections

The load-deflection relationship for both beams is shown in Figures 3.104-3.107. Figure 3.22 shows the location of the LVDT's used to measure the vertical deflection. In both beams, the load-deflection curve is linear up to the formation of flexure-shear cracking. After cracking the deflection increased significantly with additional applied load. The beams were never able to reach the peak load corresponding to initial flexure-shear cracking.

3.3.4.3 Concrete Top Fiber Strains

Figure 3.22 shows the location of the surface gages used to measure the concrete compressive strain at the top of the deck slab. The variation of the slab top strain with the applied load is given in Figures 3.108-3.111. In these figures it can be noticed that higher strains occurred in the beam with 50% debonding at the point load location even though it failed at a lower load level. Again as in the previous specimen sets, the increase in strain is associated with larger deformations due to a reduced flexural stiffness associated with flexure-shear cracking.

3.3.4.4 Stirrup Strains

Load versus stirrup strain is shown in Figures 3.112 and 3.113 for the 0% debonded beam and the 50% debonded beam respectively. The stirrup reinforcement and instrumentation is shown in Figure 3.88. It can be seen from these figures that the stirrups near the failure region reached their yield strain at gages ED4 and EA4 location when flexure-shear cracking occurred.

3.3.4.5 Longitudinal Bar Strains

The deck slab longitudinal reinforcement details and instrumentation are shown in Figure 3.114. The measured strain in the cast-in-place slab longitudinal steel is shown in Figures 3.115-3.116. Strains increased linearly with the applied load before flexure-shear cracking developed. In the 50% debonded beam substantial increase in strain without a corresponding increase in load was recorded after flexure-shear cracking occurred.

3.3.4.6 Strand Strains

The strand strains are shown in Figures 3.117-3.124. The change in the prestressing strand was relatively small. Large increase in the strain occurred when flexure cracking opened. In both beams the measured strand strain was in the linear-elastic range.

3.3.4.7 Strand Movement

Eight strands were monitored for movement in each beam as shown in Figure 3.27. In the fully bonded beam strand movement was slightly noticeable after the formation of web-shear cracking. However, significant movement did not occur until the formation of flexure-shear cracking at a load level of 245 kips. In the 50% debonded beam, strand movement was noticeable at a load level of 176 kips. Figures 3.125 and 3.126 show the load-movement behavior of the test specimen.

3.3.4.8 Failure Loads

Failure in both beams was due to lack of adequate anchorage of the prestressing strands. Excessive strand slippage caused the early opening of flexure-shear cracking

that resulted in premature failure of the beams. At failure the stirrups showed strains in excess of the yield value.

The 0% debonded beam failed at a load of 245 kips when several diagonal cracks opened in the web of the girder. Failure of the 50% debonded beam was also caused by the opening of flexure-shear cracking at a load level of 177 kips.

3.3.5 Specimen Set 5

Specimen Set 5 consisted of two pretensioned Type-I AASHTO beams composite with a 4×48 inch slab. Each beam was 308 inches long (25 feet-8 in.). One beam had all strands fully bonded while the other had 6 strands (50%) debonded near each end. The debonding points were located at 7 ft.-10 in. from ends EA and IB. The details of the strand debonding and instrumentation are given in Figure 3.127.

3.3.5.1 Cracking

The first sign of shear cracking in the fully bonded beam was in the form of a flexure-shear crack that opened near the point load at a load of 122 kips. This crack originated as a flexure crack in the bottom flange at a load of 118 kip as shown in Figure 3.128. Web-shear cracking occurred at a load of 158 kips as shown in Figure 3.129.

In the beam with 50% debonding, the first crack occurred near the debonding point, 7 ft.-10 in. from end EA, in the left shear span in the form of a flexure-shear crack at a load of 118 kips as shown in Figure 3.130. At a load of 127 kips a flexure-shear crack occurred near the point load, in both shear spans, at a distance of 134 inches from the supports. Flexure-shear cracking occurred at the debonding point in the right shear span at a load of 128 kips. Web-shear cracking occurred at a load of 156 kips at a distance of 28 inches from the left support as shown in Figure 3.131. No web-shear cracking was observed in the left shear span. The crack pattern of the 50% debonded beam at failure is shown in Figure 3.132.

3.3.5.2 Deflections

Deflection was measured using LVDT's placed at both sides of the beam as shown in Figure 3.23. The measured load-deflection relationship, for both beams, are given in Figures 3.133-3.136. In this specimen the deflection varied linearly with the applied load before flexure-shear cracking occurred. After flexure-shear cracking an increase in deflection was observed. At ultimate load, there was a large increase in deflection without corresponding increase in the applied load.

3.3.5.3 Concrete Top Fiber Strains

Concrete strains were monitored by surface gages at the top fiber of the cast-in-place slab at the point load and debonding points location as explained in Figure 3.23. The measured strain versus load curves are shown in Figures 3.137-3.140. The two beams exhibited similar behavior. At the point load location the concrete strain varied linearly with the applied load before flexure-shear cracking occurred. Significant increase in the strain was observed at failure. The limiting value of concrete strain in compression (0.003) was approached at midspan of the debonded beam. This limit was exceeded at midspan of the fully bonded beam. At the debonding point location the measured strain showed a bi-linear behavior with the applied load throughout the test for both beams. The limiting strain value was not reached at this location.

3.3.5.4 Stirrup Strains

Four of the stirrups in the shear spans of each beam were instrumented with strain gages as shown in Figure 3.141. Stirrup load-strain curves for the two beams are shown in Figures 3.142-3.144. In these beams the measured strain was very small before web-

shear cracking occurred. As expected a sudden increase in stirrup strain occurred when web-shear cracking opened. It can be noticed that none of the instrumented stirrups yielded in these tests. Due to the absence of web-shear cracking in the right shear span of the beam with 50% debonding, the stirrups near the right end (IB) did not register any significant strain.

3.3.5.5 Longitudinal Bar Strains

Three longitudinal bars in the cast-in-place slab were instrumented at two different locations as shown in Figure 3.145. The strain-load relationship is linear up to flexure-shear cracking as shown in Figures 3.146-3.149. Bi-linear behavior was shown after flexure-shear cracking. However, none of the slab longitudinal bars reached its yield strain.

3.3.5.6 Strand Strains

Figure 3.127 shows the strand debonding scheme and instrumentation for both beams. The measured strand strains for the fully bonded beam and the 50% debonded beam are shown in Figures 3.150-3.163. Before cracking the increase in strand strain was relatively small and varied linearly with the applied load. In the fully bonded beam significant increase in strain was noticed when flexure-shear cracking occurred. At failure the strands in the fully bonded beam showed strain values significantly above yield.

In the beam with 50% debonding the first flexure-shear crack formed at the debonding point in the left shear span at a load of 118 kips. As shown in Figures 3.159 and 3.160 significant increase in the strand strain occurred at the flexure-shear cracking

load. Similar behavior was shown at the debonding point in the right shear span as shown in Figure 3.163. It must be noticed that at failure the yield stress of the strand was reached near the point load location for the strands that were fully bonded.

3.3.5.7 Strand Movement

Four strands in each beam were instrumented using dial gages to detect their movement during the test as shown in Figure 3.28. In the fully bonded beam three strands moved into the girder when the load reached 172 kips. However no significant slippage occurred in this test. The load versus strand slip for the fully bonded beam is shown in Figure 3.164. In the beam with 50% debonding, all the instrumented strands were fully bonded. No movement was detected in the strands of the beam with 50% debonding until failure occurred.

3.3.5.8 Failure Loads

The fully bonded girder failed at a load of 196 kips. The beam with 50% debonding failed at a load of 157 kips. Failure occurred by crushing of the top concrete fibers of the cast-in-place slab as shown in Figures 3.165 and 3.166. This indicated a typical flexure failure following yielding of the strands.

3.4 Summary

This chapter contains the description and results of the tests conducted in Phase 2 of the research program. The behavior of simply supported precast pretensioned bridge girders with debonded strands and a cast-in-place composite slab was evaluated. The simply supported girders were tested under a single monotonic concentrated load.

Crack patterns, load-deflection curves, stirrup strains, strand strains, strand slip, longitudinal non-prestressed reinforcement strains, and the concrete surface strain at the top of the cast-in-place slab were presented.

In all the specimen sets large deflection were observed in the beams with debonded strands even though the failure loads of the debonded beams were lower than those of the fully bonded. The measured stirrup strains showed that, in each specimen set higher strains were observed in the debonded beam.

Prior to cracking, the change in the prestressing strand strains was small and proportional to the change in the applied load. Sudden increase in the strand strains occurred when flexure-shear cracking developed. Measurements of the compressive concrete strain at the top of the cast-in-place slab revealed that higher strains occurred in the beams with debonded strands. It was also noticed that flexure-shear cracking in the beams with debonded strands developed at the debonding points. This confirmed the earlier findings in beam with debonded strands.

Both the debonded and the fully bonded beams of Specimen Sets 2, 3, and 4 failed due to inadequate anchorage of the required force in the prestressing strands. Cracking in the bottom flange near the ends of the girders disturbed the development length and

caused bond and anchorage failure. This type of failure did not occur in Specimen Set 5 in which higher development lengths were provided for the prestressing strands in spite of the presence of web-shear and flexure-shear cracking.

The next chapter contains the analysis of the results obtained from testing the simply supported members under the effect of a single concentrated load. Summary, conclusions, and future work are presented in chapter 5.

CHAPTER 4

ANALYSIS OF EXPERIMENTAL RESULTS

4.1 Introduction

In this chapter the behavior of the simply supported beams reported in Chapter 3 is evaluated and the results compared with predicted values from current ACI/AASHTO provisions. The predicted web-shear and flexure-shear cracking loads at the critical section and at the initial crack location are compared to the observed test values. The predicted shear and flexural failure loads using ACI/AASHTO provisions are also compared with the corresponding test results.

An evaluation of the current ACI/AASHTO requirement for strand development length is presented in this chapter.

4.2 Effective Strand Stress

Strains in the prestressing strands were measured by electrical resistance strain gages. The measured strains were used to calculate stress using the corresponding stress-strain behavior given in Chapter 3. Table 4.1 gives the effective strand stress at the time of the tests. These values were used in all the calculations performed in this investigation. These values represent the average of the measured strand stress at different sections along the beam. The effective strand stress in the beams with debonded strands was higher than in the fully bonded. It is seen that, the prestress

losses due to concrete elastic shortening and time-dependent creep deformations resulting from the prestressing were less in the beams with debonded strands. These values are in close agreement with predicted values from PCI method [1975] calculated at midspan sections.

4.3 Web-shear Cracking

Web-shear cracking loads were calculated at the critical section, $H/2$ away from the support face and at the initial crack location. H is the total depth of the composite beam. The ACI/AASHTO equation given below is used to calculate the web cracking strength:

$$V_{cw} = (3.5 \sqrt{f'_c} + 0.3 f_{pc}) b_w d \quad (4.1)$$

where:

V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in the web

f'_c = compressive strength of concrete

f_{pc} = compressive stress (after allowance for all prestress

losses) at the centroid of the composite section, or at the

junction of the web and flange when centroid lies within the flange,

due to both prestressing and the moment resisted by the precast member acting alone

b_w = web width of the girder

d = distance from the extreme compression fiber to the centroid of the prestressing tension reinforcement ($\geq 0.8 H$)

Table 4.1
Effective Strand Stress

Specimen Set	Beam	Effective Strand Stress (ksi)
1	50% Debonded	133.7
	Fully Bonded	126.1
2	50% Debonded	153.7
	Fully Bonded	140.7
3	67% Debonded	164.0
	Fully Bonded	160.0
4	50% Debonded	164.1
	Fully Bonded	161.1
5	50% Debonded	168.0
	Fully Bonded	151.7

Table 4.2
Web-Shear Cracking Loads at Critical Section (H/2)

Specimen Set	Beam	Predicted Load (kips)	Observed Load (kips)	obs/pred
1	50% Debonded	88	120	1.36
	Fully Bonded	105	150	1.43
2	50% Debonded	87	114	1.31
	Fully Bonded	97	130	1.34
3	67% Debonded	82	110	1.34
	Fully Bonded	100	152	1.52
4	50% Debonded	191	176	0.92
	Fully Bonded	241	160 (245)	0.66 (1.02)
5	50% Debonded	111	156	1.41
	Fully Bonded	136	158	1.16

Table 4.3
Web-Shear Cracking Loads at Initial Crack Location

Specimen Set	Beam	Predicted Load (kips)	Observed Load (kips)	obs/pred
1	50% Debonded	88	120	1.36
	Fully Bonded	105	150	1.43
2	50% Debonded	86	114	1.33
	Fully Bonded	98	130	1.33
3	67% Debonded	77	110	1.43
	Fully Bonded	100	152	1.52
4	50% Debonded	179	176	0.98
	Fully Bonded	241	160 (245)	0.66 (1.02)
5	50% Debonded	112	156	1.39
	Fully Bonded	137	158	1.15

Tables 4.2 and 4.3 show that the ACI/AASHTO equations gave conservative estimates of the web-shear cracking loads for the I-beams regardless of the amount of debonding. However, the degree of conservatism decreased in regard to the actual web-shear cracking capacity of the box beams. The early web-shear cracking in the fully bonded box-beam specimen was in part determined to be due to the difference in the thickness of the walls on the two sides of the beam. After testing, the thickness of the wall was determined to be 4.5 inches on the side where cracks were first observed while the other had a thickness of 5.5 inches. The values in parentheses in Tables 4.2 and 4.3 were the observed loads when web-shear cracking occurred in the thicker wall.

4.4 Flexure-shear Cracking

The flexure-shear cracking capacity of prestressed beams according to the current ACI/AASHTO Codes, V_{ci} , is given by:

$$V_{ci} = 0.6 \sqrt{f'_c} b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \quad (4.2)$$

where:

V_d = shear force at section due to unfactored dead load

V_i = factored shear force at section due to externally applied loads

M_{max} = maximum factored bending moment at section due to externally applied loads

M_{cr} = moment causing flexural cracking at section due to externally applied loads

$$M_{cr} = \frac{I}{y_t} (6\sqrt{f'_c} + f_{pe} - f_d)$$

I = moment of inertia of composite section

y_t = distance from centroid of composite section to extreme tension fiber

f_{pe} = compressive stress, due to effective prestressing, at extreme fiber of section where tensile stresses are caused by externally applied loads

f_d = stress due to unfactored dead load, at extreme fiber of section where tensile stresses are caused by externally applied loads

The predicted flexure-shear cracking loads were compared to the observed values in Table 4.4. The critical section for flexure-shear cracking was taken at the location where these cracks occurred.

The ACI/AASHTO equations gave unconservative predictions of the flexure-shear cracking loads for the debonded beams. For the fully bonded beams good agreement

was achieved when no excessive strand slippage occurred (Specimen Sets 1, 2, and 5).

Table 4.4
Flexure-shear Cracking Loads at Initial Crack Location

Specimen Set	Beam	Predicted Load (kips)	Observed Load (kips)	obs/pred
1	50% Debonded	135	113	0.84
	Fully Bonded	160	165	1.03
2	50% Debonded	190	120	0.63
	Fully Bonded	173	180	1.04
3	67% Debonded	143	102	0.71
	Fully Bonded	206	158	0.77
4	50% Debonded	336	176	0.52
	Fully Bonded	304	240	0.79
5	50% Debonded	111	118	1.06
	Fully Bonded	121	122	1.01

Table 4.5 shows the predicted flexure-shear cracking loads at the critical section. The theoretical critical section for flexure-shear cracking in the fully bonded beams, in all specimen sets, was at the point of application of the external load. The critical section for the beam with 50% debonding in Specimen Set 1 was at the second debonding point, 62.5 inches from the centerline of the end support. In Specimen Set 2 the critical section for flexure-shear cracking was also at the second debonding point, a distance of 71 inches from the end support. The critical section for the beam with 67% debonding in Specimen Set 3 was at the first debonding point, 42 inches from the end support. In Specimen Set 4 the critical section was at the point load. For Specimen Set 5 the critical section was at the debonding points, which were located at 84 inches from

the end supports. The results of Table 4.5 indicated that better agreement with test results was obtained for flexure-shear cracking loads. However, the predicted cracking loads for the beams with debonded strands of Specimen Sets 1, 2, 3, and 4 were unconservative.

The predicted flexure-shear cracking loads for the debonded beams at the critical section considering only the strands which had developed the full prestressing force are given in Table 4.6. The theoretical critical section for all beams was at the point load. Improved agreement with the observed flexure-shear cracking loads was obtained with this modification. It must be noted that, the beam with 50% in Specimen set 5 developed the full prestressing force at the point load. The critical section for this beam was at the debonding point.

Table 4.5
Flexure-shear Cracking Loads at Critical Section

Specimen Set	Beam	Predicted Load (kips)	Observed Load (kips)	obs/pred
1	50% Debonded	135	113	0.84
	Fully Bonded	131	165	1.26
2	50% Debonded	140	120	0.86
	Fully Bonded	152	180	1.18
3	67% Debonded	140	102	0.73
	Fully Bonded	166	158	0.95
4	50% Debonded	251	176	0.70
	Fully Bonded	269	240	0.89
5	50% Debonded	111	118	1.06
	Fully Bonded	101	122	1.21

Table 4.6
Flexure-shear Cracking Loads at Critical Section
with a Reduced Number of Effective Strands

Specimen Set	Beam	Predicted Load (kips)	Observed Load (kips)	obs/pred
1	50% Debonded	95	113	1.19
	Fully Bonded	131	165	1.26
2	50% Debonded	113	120	1.06
	Fully Bonded	152	180	1.18
3	67% Debonded	116	102	0.88
	Fully Bonded	166	158	0.95
4	50% Debonded	209	176	0.84
	Fully Bonded	269	240	0.89
5	50% Debonded	111	118	1.06
	Fully Bonded	101	122	1.21

4.5 Shear Strength

The shear capacity of the test beams was calculated by adding the contribution of the web reinforcement to the lower value of V_{cw} and V_{ci} at a given design section. The shear resistance provided by the stirrup reinforcement is given by:

$$V_s = \frac{A_v f_y d}{s} \quad (4.3)$$

where:

V_s = nominal shear strength provided by web reinforcement

f_y = yield strength of web reinforcement

A_v = cross-sectional area of the stirrups

d = distance from extreme compression fiber to the centroid of prestressing
tension reinforcement ($\geq 0.8 H$)

s = stirrup spacing

At any section along all the beams tested the contribution of the stirrup reinforcement consisted of #3 U, Grade 60 stirrups at 4 inches on centers. This yields a V_s term of 110 kips. All the beams were designed to ensure that shear failure would not occur. The predicted shear failure loads are given in Table 4.7. It can be seen that the beams tested in this study did not reach the predicted shear capacity. It also needs to be pointed out that none of the specimens failed in shear. However, it should also be observed that the considerable amount of stirrup reinforcement in these beams $r f_y = \frac{A_v}{b_w s} f_y = 660$ psi for the I-beams and 396 psi for the box beams, did not help to prevent premature failures in the case of Specimen Sets 2, 3, and 4 where an adequate anchorage of the longitudinal tension chord was not provided and the debonded beam in Specimen Set 1. It must be remarked that both girders in Specimen Set 1 were tested to failure, even though slab failure due to inadequate slab transverse reinforcement occurred prior to beam failure. In the case of Specimen Set 5 the mode of failure was flexure and both beams reached the expected capacity as it is discussed in the next section.

Table 4.7
Shear Failure Loads

Specimen Set	Beam	Predicted Load (kips)	Observed Load (kips)	obs/pred	Failure Mode
1	50% Debonded	268	148	0.55	Slab failure
	Fully Bonded	297	225	0.76	
2	50% Debonded	267	172	0.64	Strand anchorage
	Fully Bonded	288	194	0.67	
3	67% Debonded	259	118	0.46	Strand anchorage
	Fully Bonded	292	160	0.55	
4	50% Debonded	389	177	0.46	Strand anchorage
	Fully Bonded	441	245	0.56	
5	50% Debonded	317	157	0.50	Flexure Flexure
	Fully Bonded	319	196	0.61	

4.6 Flexural Strength

All the pretensioned beams in Specimen Sets 2, 3 and 4 tested in this investigation failed due to bond and anchorage loss. The girders in specimen Set 1 experience slab failure due to inadequate slab reinforcement prior to actual beam failure. However, they were loaded beyond slab failure until girder failures occurred. The results are included herein for informational purposes only. The combined presence of shear and flexural cracks in the bottom flange reduced the available anchorage length of the strands causing failure of the pretensioned beams. As noted in the previous section, even the close stirrup spacing provided was not adequate to prevent this failure mode. This was not the case for Specimen Set 5. The development length specified by the ACI Building Code [1989] is given by:

$$l_d = \frac{f_{se}}{3} d_p + (f_{ps} - f_{se}) d_p \quad (4.3)$$

where:

l_d = development length of the prestressing strand

f_{ps} = stress in prestress reinforcement at nominal strength

f_{se} = effective stress in prestressed reinforcement

d_p = nominal diameter of prestressing strand

The above expression was also adopted by AASHTO Specifications [1989]. Florida Department of Transportation has suggested a revision in the ACI/AASHTO specifications pertaining to the development of prestressing strands based on an investigation carried out by El Shahawy and Batchelor in 1991. Based on the results obtained from testing 33 AASHTO Type II pretensioned beams, El Shahawy and

Batchelor concluded that the ACI/AASHTO requirement on development length of prestressing strands is not adequate. They recommended that the development length should be increased to 1.69 times the ACI/AASHTO values, if the full flexural strength of the beam is to be developed. The development length for the fully bonded strands in both fully bonded and debonded beams in each specimen set calculated based upon the ACI/AASHTO Codes and FDOT Specifications are summarized in Table 4.8. The values shown in parenthesis represent the requirement of doubling the development length for strands not bonded all the way to the end of the member.

Table 4.8
Development Length

Specimen Set	Beam	Development Length (inch)	
		ACI/AASHTO	FDOT
1	50% Debonded	90.4 (180.8)	152.8
	Fully Bonded	93.0	157.2
2	50% Debonded	83.8 (167.6)	141.6
	Fully Bonded	88.1	149.0
3	67% Debonded	80.3 (160.6)	135.7
	Fully Bonded	81.7	138.1
4	50% Debonded	80.3 (160.6)	135.7
	Fully Bonded	81.3	137.4
5	50% Debonded	79.0 (158)	133.5
	Fully Bonded	84.4	142.6

The available development length of the fully bonded strands in Specimen Sets 1, 2, 3, and 4 was 100 inches. The available development length of the fully bonded strands

in Specimen Set 5 was 154 inches. The number of effective strands at the location of the maximum bending moment was calculated based on the available embedment length of the strand. A linear relationship was used to find the effective area of the strands which were not fully developed at the point of maximum moment. In this analysis, the development length specified by ACI/AASHTO Specifications was doubled for debonded strands as required. The number of strands considered effective in computing the ultimate moment capacity of each of the girders tested is given in Table 4.9.

The flexural strength of each girder was calculated with strain compatibility analysis using only the effective strands given in Table 4.9. The calculated flexural failure loads were compared to the observed test values in Table 4.10.

The results of Specimen Sets 1, 2, and 3 indicated that the predicted failure load based on FDOT development length requirements was in general more conservative than the values given by current ACI/AASHTO specifications. For debonded I-beams the ACI/AASHTO predicted load is in better agreement with the test results of Specimen sets 1, 2, and 3. The box girder specimens failed at load levels which are below the loads predicted according to both FDOT and AASHTO recommendations. The beams in Specimen Set 5 showed that both approaches yield comparable conservative estimate of the failure capacity of I-beam girders.

Table 4.9
Number of Effective Strands

Specimen Set	Beam	No. of Effective Strands	
		ACI/AASHTO	FDOT
1	50% Debonded	7.20	5.30
	Fully Bonded	12.0	7.60
2	50% Debonded	7.00	5.50
	Fully Bonded	12.0	8.10
3	67% Debonded	5.64	4.89
	Fully Bonded	12.0	8.69
4	50% Debonded	11.9	9.58
	Fully Bonded	20.0	14.6
5	50% Debonded	8.28	8.70
	Fully Bonded	12.0	12.0

Table 4.10
Flexural Failure Loads

Specimen Set	Beam	Predicted Load (kips)	Observed Load (kips)	obs/pred
1	50% Debonded ACI/AASHTO FDOT	141.0	148.2	1.05
		105.1	148.2	1.41
	Fully Bonded ACI/AASHTO FDOT	228.6	225.4	0.99
		148.4	225.4	1.52
2	50% Debonded ACI/AASHTO FDOT	147.3	172.0	1.17
		118.0	172.0	1.46
	Fully Bonded ACI/AASHTO FDOT	245.0	194.0	0.79
		169.0	194.0	1.15
3	67% Debonded ACI/AASHTO FDOT	119.0	118.0	0.99
		104.4	118.0	1.13
	Fully Bonded ACI/AASHTO FDOT	242.0	160.0	0.66
		178.5	160.0	0.90
4 *	50% Debonded ACI/AASHTO FDOT	250.7	177.0	0.71
		205.5	177.0	0.86
	Fully Bonded ACI/AASHTO FDOT	408.8	245.0	0.60
		303.5	245.0	0.81
5	50% Debonded ACI/AASHTO FDOT	126.0	157.0	1.25
		132.0	157.0	1.19
	Fully Bonded ACI/AASHTO FDOT	180.0	196.0	1.09
		180.0	196.0	1.09

* Predicted capacity for debonded box beam in Specimen Set 4 neglecting the contribution of debonded strands $P=214$ kips, $obs/pred=0.83$.

4.7 Summary

The shear and flexural behavior of the specimens reported in Chapter 3 was evaluated in this chapter. Comparison of the predicted and observed results indicated that the ACI/AASHTO Codes gave conservative estimates of the web-shear cracking capacity for both the debonded and fully bonded I-beams. However, they gave adequate estimates for the web-shear cracking capacity of the debonded box girder and unconservative for the fully bonded one. It can be concluded that strand debonding did not significantly influence the web-shear cracking capacity of the AASHTO I-beams as predicted by current ACI/AASHTO specifications. Further work may be needed to verify the findings in the case of box sections. It was noted that flexure-shear cracking developed in the debonded beams earlier than predicted by the ACI/AASHTO procedures. All flexure-shear cracks in the debonded beams originated at or near the debonding points. Reasonable agreement with the test results was obtained when the strands which were not fully developed at the critical section were not considered in the analysis. All beams where anchorage failure of the fully bonded strands was noted failed at loads lower than the lesser of the predicted shear and flexural failure loads. It must also be noted that close stirrup spacing in these beams did not eliminate the anchorage type failure.

Excessive strand slip after shear cracking had occurred, preceded the failure of both beams in Specimen Sets 2, 3, and 4. It was concluded that the current ACI/AASHTO provisions for development length of fully bonded prestressing strands in pretensioned beams was not adequate in the presence of shear cracking. In the particular case of

web-shear cracking excessive slippage occurred when this crack penetrated into the transfer length of the strand. A web-shear crack that does not penetrate into the transfer length of the strand is not critical as shown by the fully bonded beam in Specimen Set 2. However, the requirement for doubling the development length for strands not bonded all the way to the end of the member required by these codes resulted in a conservative estimate of the flexural failure load when used in the calculation of the effective number of strands at the critical section except for the box beam specimen.

Both beams in Specimen 5 failed at loads higher than the calculated ultimate loads based on flexural capacity. The fully bonded prestressing strands in Specimen 5 had development length of $1.9 l_d$ with l_d based on the current ACI/AASHTO provisions. This available length was also greater than the $1.7 l_d$ recommended by the FDOT.

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 Summary

This report presents the results of an experimental investigation conducted to evaluate the effect of strand debonding on the behavior of simply supported pretensioned concrete beams. The test program involved five specimen sets. Each specimen set consisted of two beams, one of the beams had all strands fully bonded to the surrounding concrete. The other had some strands debonded towards the ends. Specimen Sets 1, 2, 3 and 5 were Type-I AASHTO girders composite with a 4×48 inch cast-in-place concrete slab. Specimen Set 4 consisted of two Indiana State Type CB-27 box girders with a 4×36 inch cast-in-place slab. All beams were tested simply supported under the effect of a single monotonic concentrated load. The flexural and shear behavior of the debonded beams is compared to that identical fully bonded beams in each set.

5.2 Conclusions

1. The current ACI/AASHTO equation for web-shear cracking of composite prestressed beams gave conservative estimates of the web-shear cracking loads of both fully bonded and debonded I-beams. Slightly unconservative results were obtained for the box girders.

2. Flexure-shear cracking developed in the debonded beams of Specimen Sets 1, 2, 3, and 4 earlier than predicted by the ACI/AASHTO codes. It was noted that flexure-shear cracking occurred at the debonding points. It can be concluded that, based on the predicted values of current specifications strand debonding reduces flexure-shear cracking capacity of pretensioned beams compared to that of fully bonded members. Good agreement with the observed flexure-shear cracking loads at a given section was obtained when the strands which were not fully developed based on current specifications, were not considered effective in the calculation of the flexure-shear cracking load. The flexure-shear cracking load in the debonded beam of Specimen Set 5, with $1.8 l_d$ anchorage length for the fully bonded strands, was adequately predicted by the current ACI/AASHTO recommendations. The debonded strands developed the required prestressing force at the point load. In this beam no slippage of the fully bonded strands was observed. The fully bonded beams, in Specimen Sets 3 and 4, where excessive strand slippage was observed prior to flexure-shear cracking, showed significant reduction in the flexure-shear cracking loads with respect to predicted values. In these specimens, web-shear cracking formed prior to flexure-shear cracking and penetrated into the girder bottom flange within 20 inches from the support centerline. Thus affecting the transfer length of the strands. The predicted cracking loads were in good agreement with the test results of the fully bonded beams in Specimen Set 1, 2, and 5 where flexure-shear cracking opened prior to strand movement due to web-shear cracking affecting the strand transfer length.

3. In spite of the close stirrup spacing in the longitudinal direction, #3 U at 4 inches on centers ($r f_y = 660$ psi for the I-beams and 396 psi for the box beams), anchorage failures were observed in Specimen Sets 2, 3 and 4 and the debonded beam in Specimen Set 1. Only when an anchorage length equal $1.8 l_d$ was provided in Specimen Set 5, the flexural-shear capacity was achieved in both debonded and fully bonded beams.
4. The fully bonded beams of the first four specimens failed before the predicted flexural ultimate loads were reached. In Specimen Sets 2, 3 and 4 failure was preceded by slippage of the prestressing strands. It can be concluded that the strand development length specified by the ACI/AASHTO codes for fully bonded strands is not adequate if web-shear cracking penetrates into the transfer length of the strand, or if flexure-shear cracking occurs within the current full anchorage length, l_d of the strand. The beams in Specimen set 5 with development length available for the fully bonded strands, greater than that required by the ACI/AASHTO codes failed at loads higher than the predicted values. However, reasonable agreement with test results in the debonded beams of Specimen Sets 1, 2, 3, and 5 (I-beams) was obtained, when the number of the effective strands was based on the current ACI/AASHTO provision for doubling the required development length of debonded strands.
5. When the requirement of doubling the development length for debonded strands was used in the calculation of the flexural failure loads, unconservative estimate was obtained for the debonded box girder. Neglecting entirely the contribution of

the debonded strands also resulted in unconservative estimate based on current ACI/AASHTO requirements.

6. Adequate anchorage length for the prestressing strand in pretensioned beams is of critical importance in reaching the full ultimate capacity both in flexure and shear. Excessive strand slippage in beams with inadequate anchorage length upon formation of web-shear cracks penetrating into the transfer length of the strand or flexure-shear cracks within the anchorage length of the strand resulted in a significant reduction in the load carrying capacity of the girders with respect to predicted values based on current ACI/AASHTO recommendations. These tests indicated that the required development length of Grade 270, seven-wire strands, 0.5 inch diameter is greater than required by the ACI/AASHTO equation if shear cracking as described above occurs within the anchorage length.
7. The flexure and shear design of both bonded and debonded pretensioned I-beams, where the flexural capacity controls, based on current ACI/AASHTO design provisions would be adequate provided that the fully bonded strands in the member have anchorage length of at least $1.7 l_d$. This recommendation is based on the results of the fully bonded beam in Specimen Set 3 as a lower bound. This value of $1.7 l_d$ is also in agreement with the finding in the FDOT study. In debonded members the $1.7 l_d$ requirement should be applied to both fully bonded and debonded strands.
8. The findings from this study indicate that the degree of conservatism decreases as the percentage of debonding increases. It is recommended that no more than

67% of the strands be debonded. The current limit of 50% was shown to be conservative provided the anchorage length of the fully bonded strand is at least $1.7 l_d$, with l_d based on current ACI/AASHTO requirements.

9. Findings of this study indicate that no limitation would seem to be required in regard to the number of strands debonded on a given row. However, to avoid large stress concentration it is recommended to stagger the debonding with a total percentage of strands debonded in the girder not greater than 67%.

5.3 Future Work

The current ACI/AASHTO Specifications yielded unconservative estimates of the flexural failure loads of both beams in the box girder specimen set. Further research is needed to examine the behavior of simply supported pretensioned box beams with debonded strands.

The I-beam specimens in this study were all designed so that flexure would control the member capacity. To fully determine the adequacy of current AASHTO design recommendations for shear, it would be necessary to observe experimentally the behavior of bonded and debonded specimens designed to fail in shear with an available anchorage length of $1.7 l_d$ for the fully bonded strands.

The current ACI/AASHTO provisions for the development length of fully bonded strands appear inadequate. Further experimental research is needed to determine the adequate development length for both fully bonded and debonded strands in pretensioned concrete beams.

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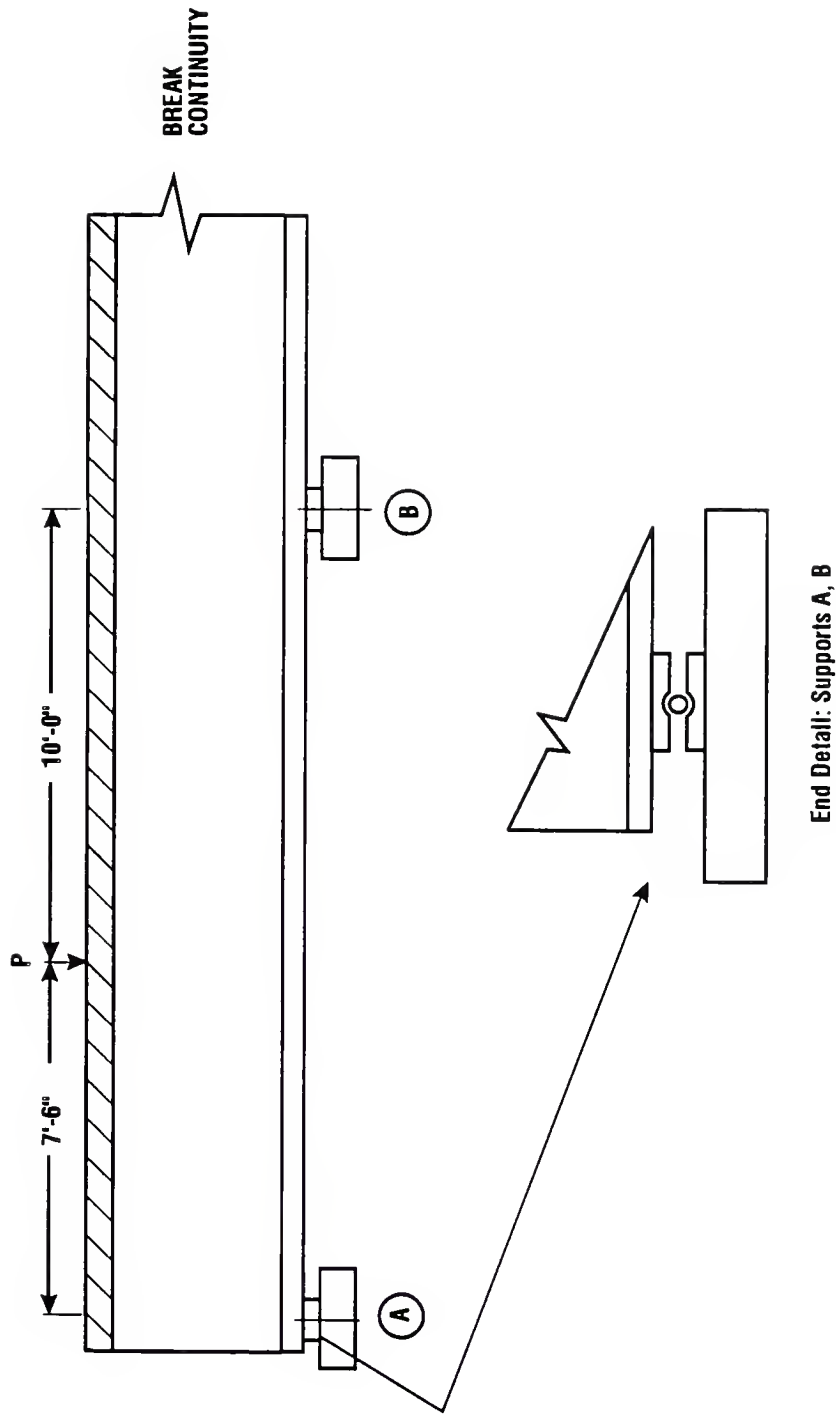


Figure (3.1) Simply Supported Tests Setup for Specimen Sets 1, 2, 3 and 4.

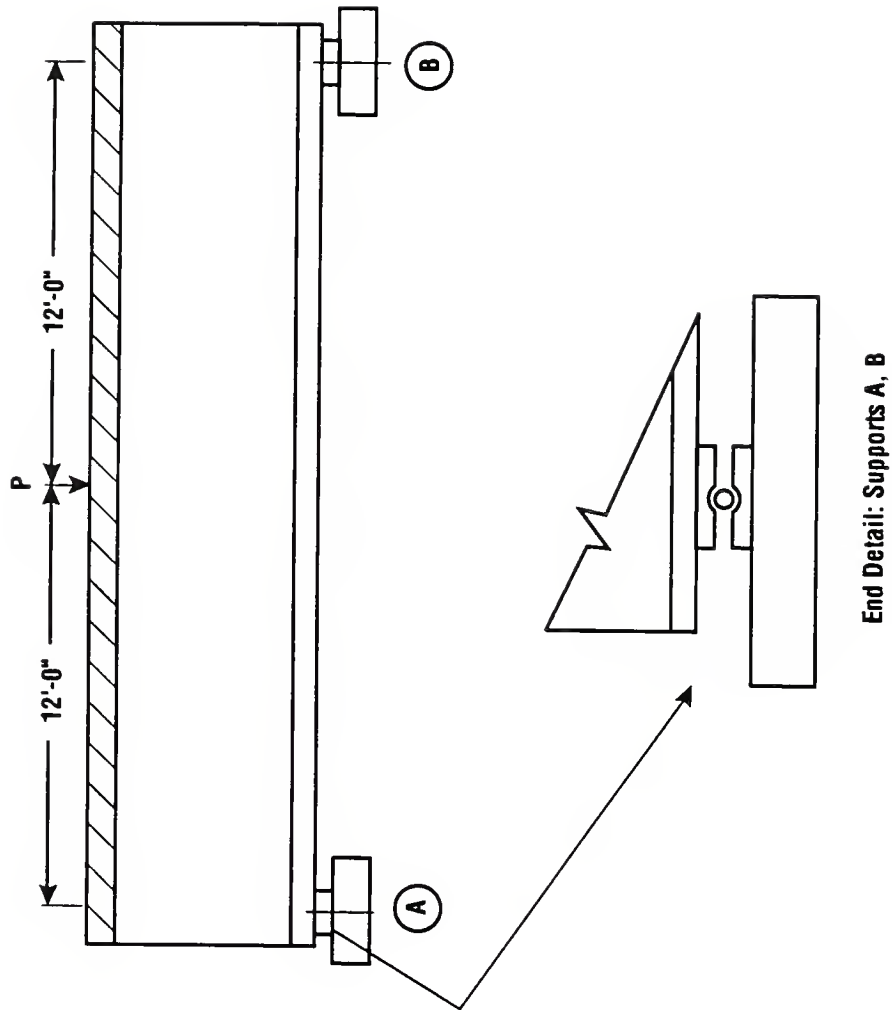
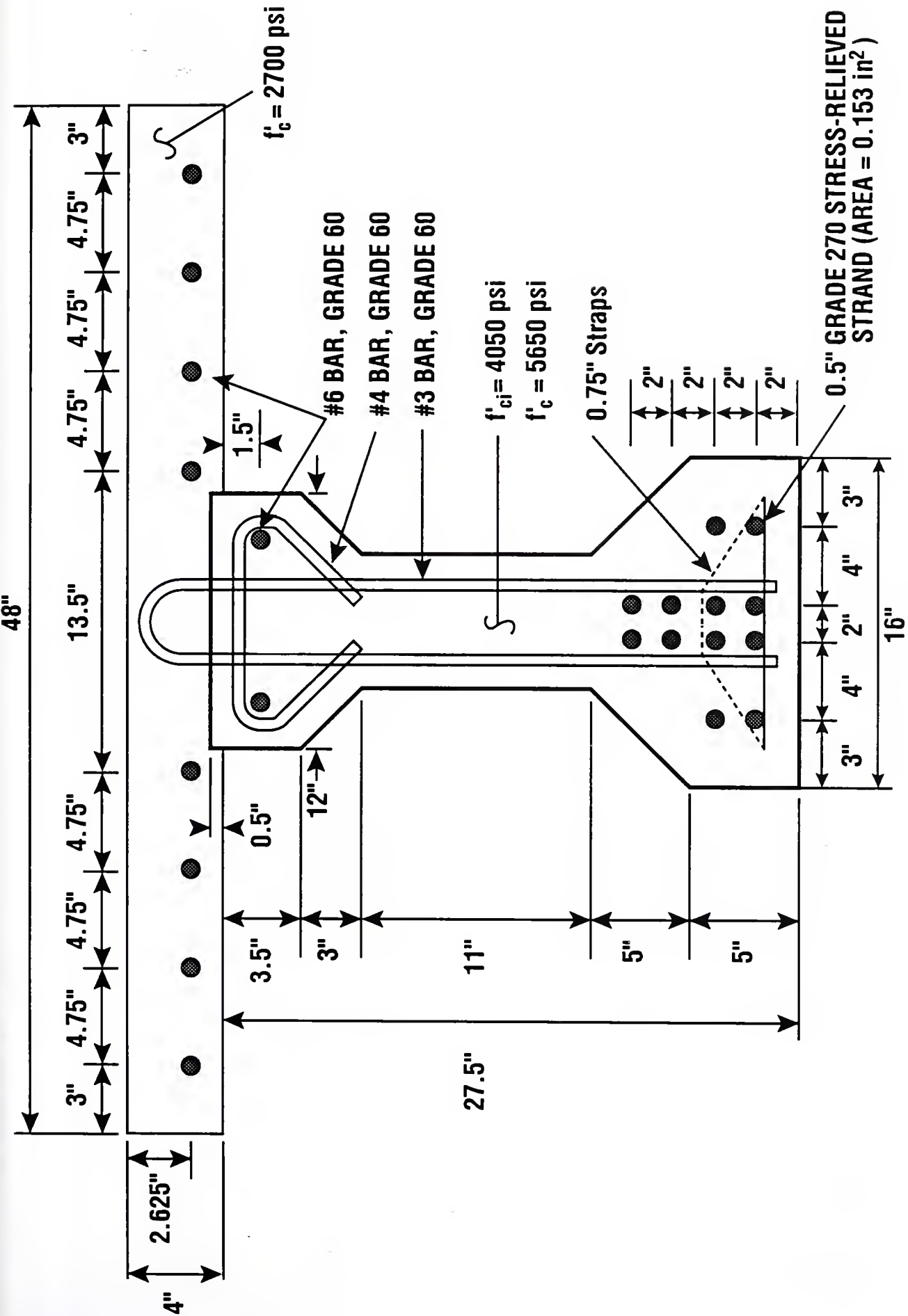
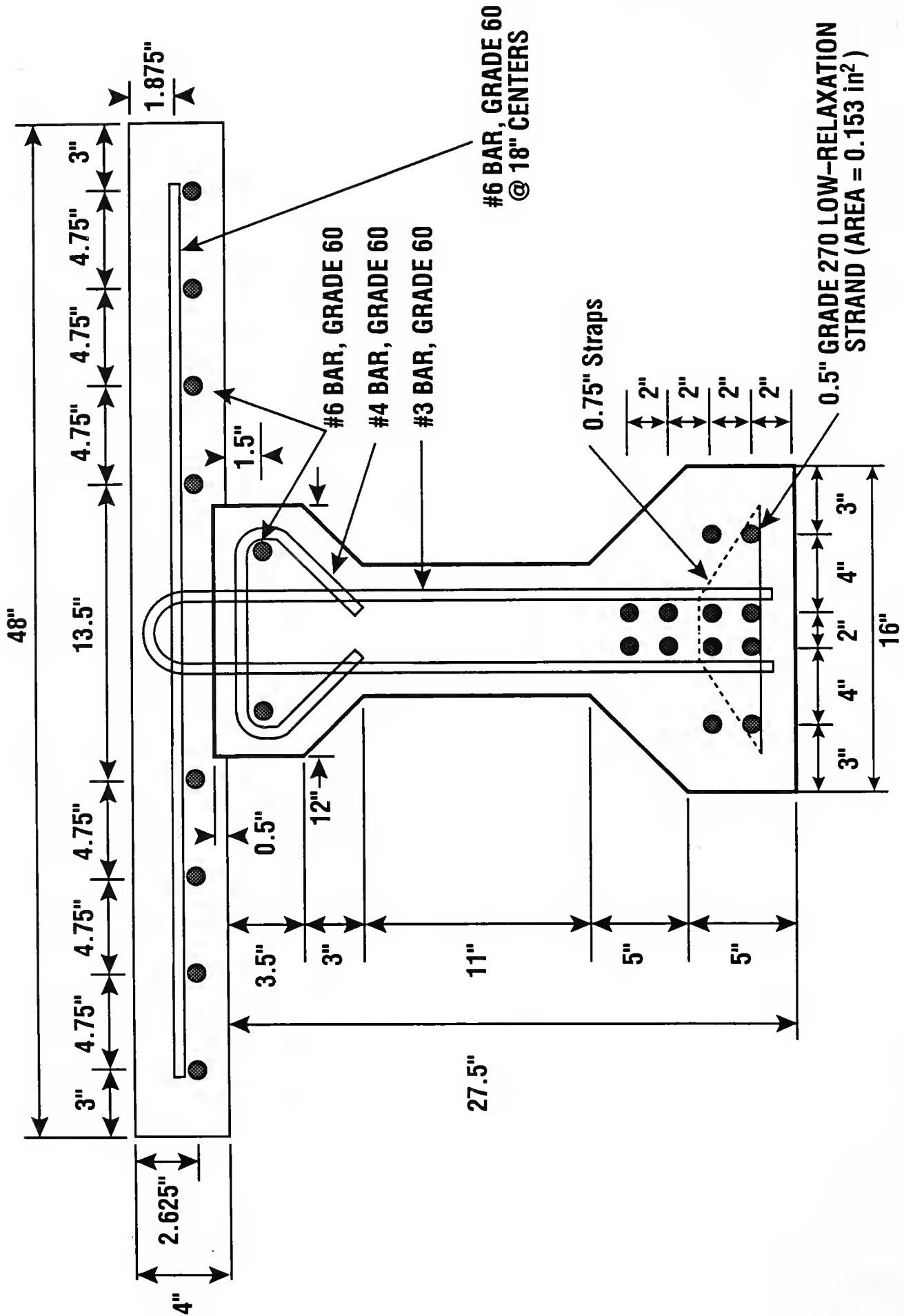


Figure 3.2 Simply Supported Tests Setup for Specimen Set 5.



TYPE I

Figure (3.3) Composite Girder Cross-section and Detail for Specimen Set 1.



TYPE I

Figure (3.4) Composite Girder Cross-section and Detail for Specimen Sets 2, 3 and 5.

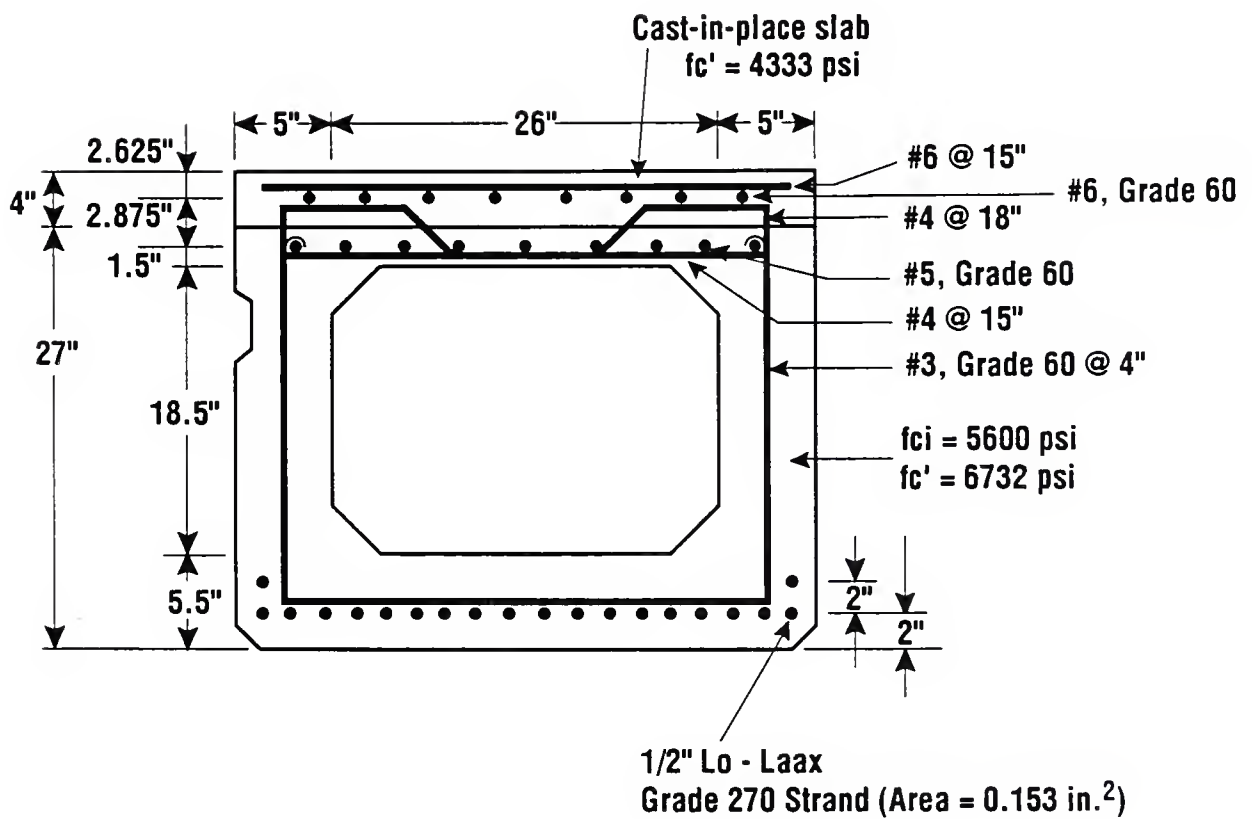


Figure (3.5) Composite Girder Cross-section and Details for Specimen Set 4.

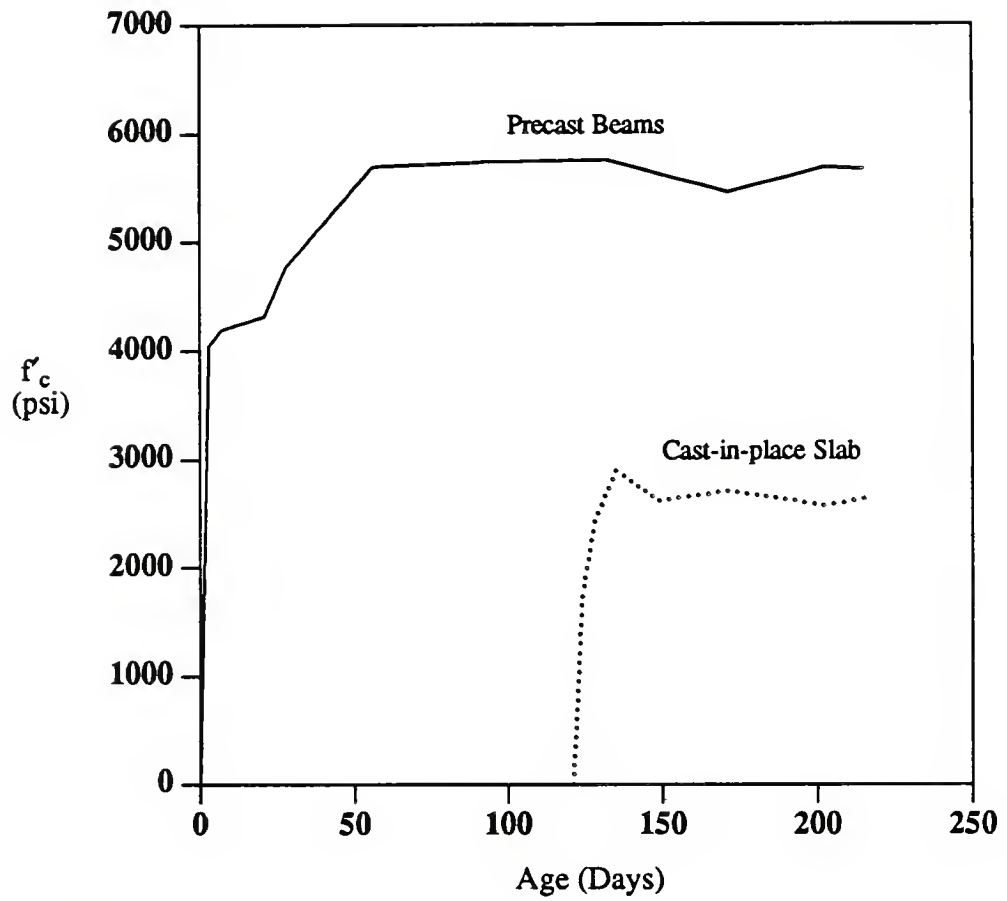


Figure 3.6 Variation of Uniaxial Compressive Strength
of Concrete With Age
Specimen Set 1

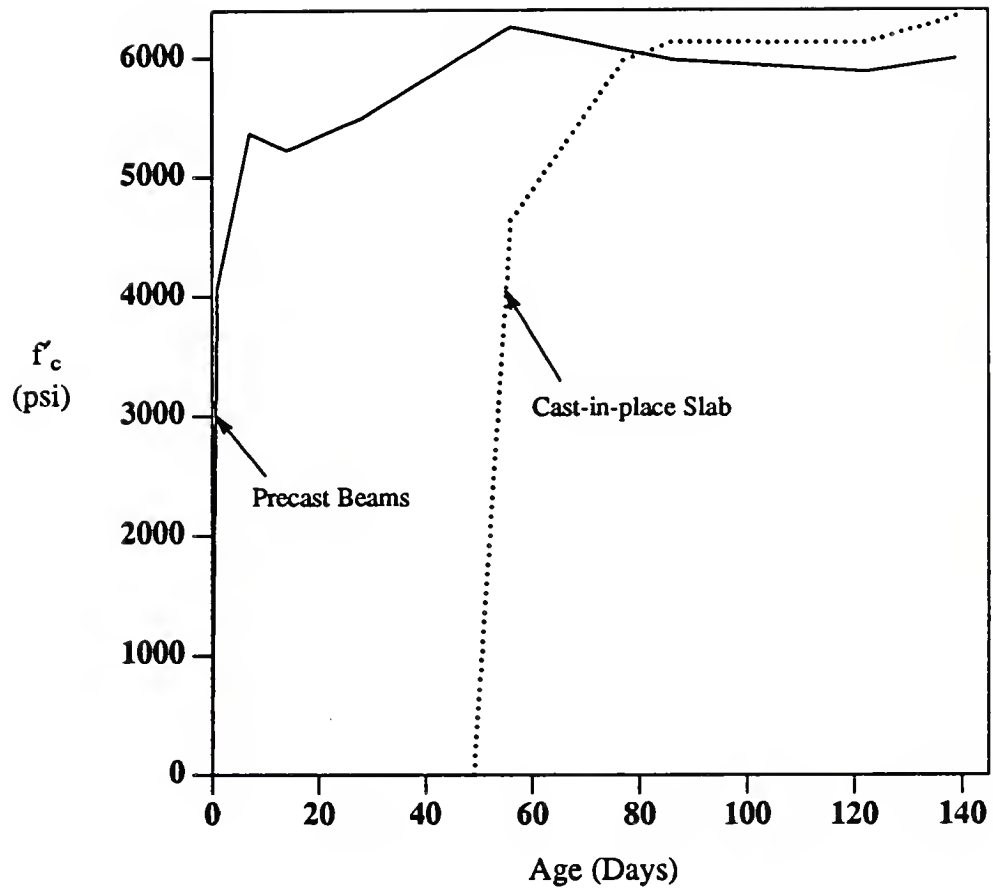


Figure 3.7 Variation of Uniaxial Compressive Strength
of Concrete With Age
Specimen Set 2

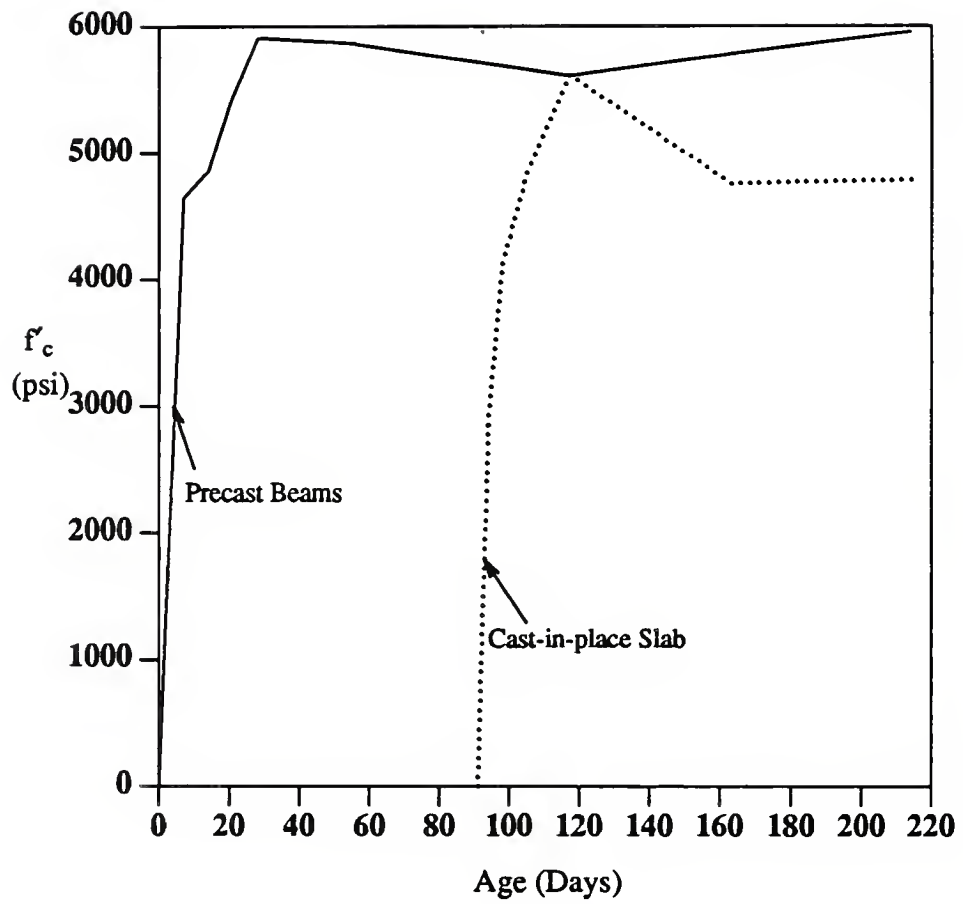


Figure 3.8 Variation of Uniaxial Compressive Strength
of Concrete With Age
Specimen Set 3

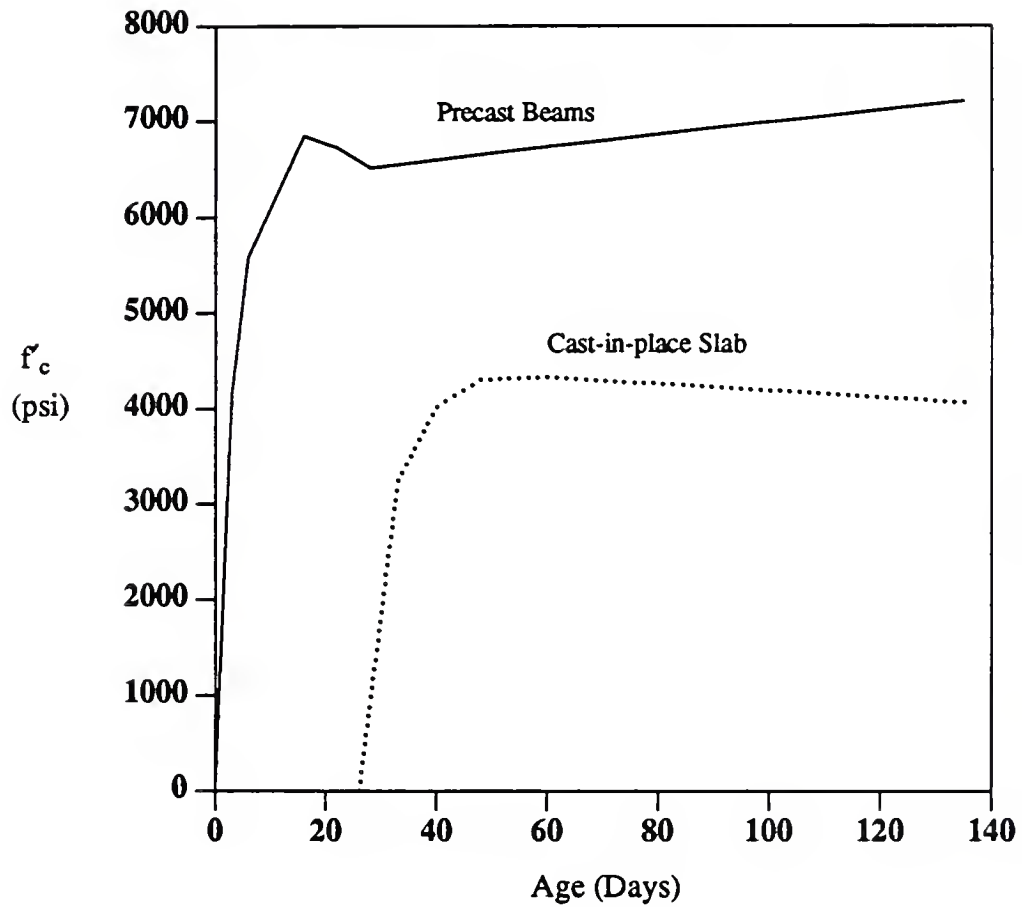


Figure 3.9 Variation of Uniaxial Compressive Strength
of Concrete With Age
Specimen Set 4

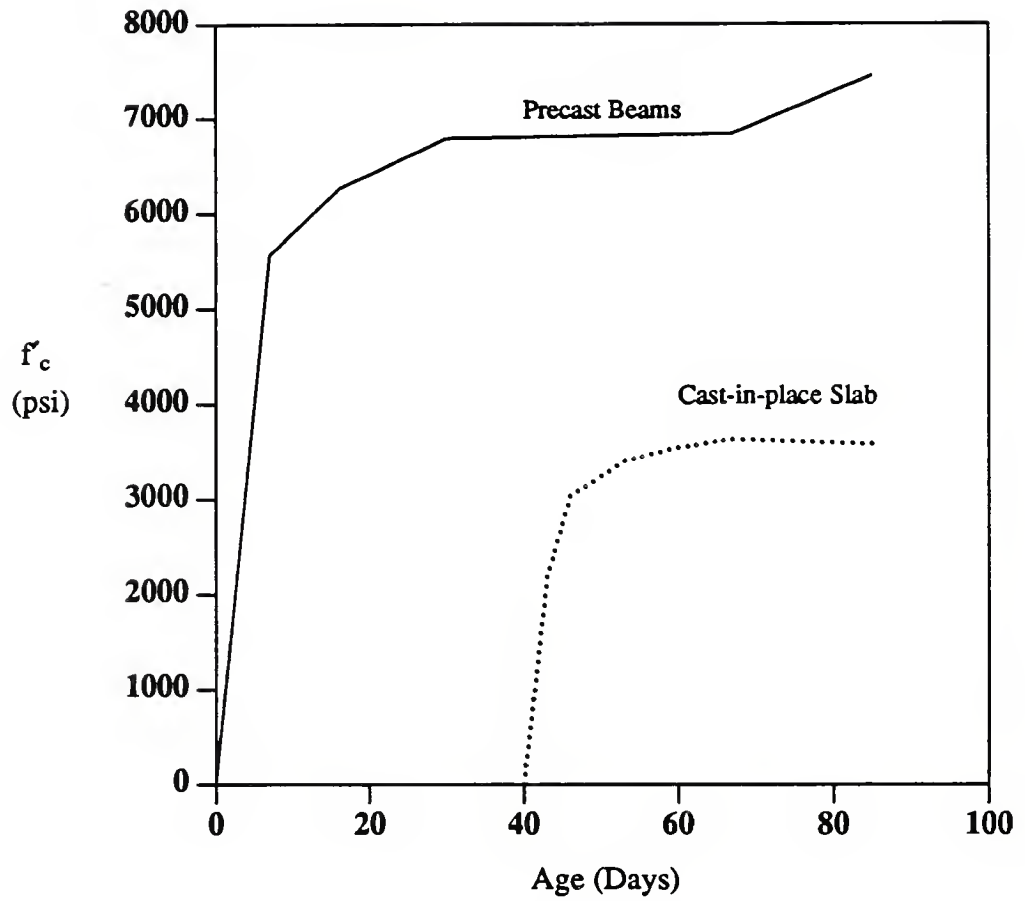


Figure 3.10 Variation of Uniaxial Compressive Strength
of Concrete With Age
Specimen Set 5

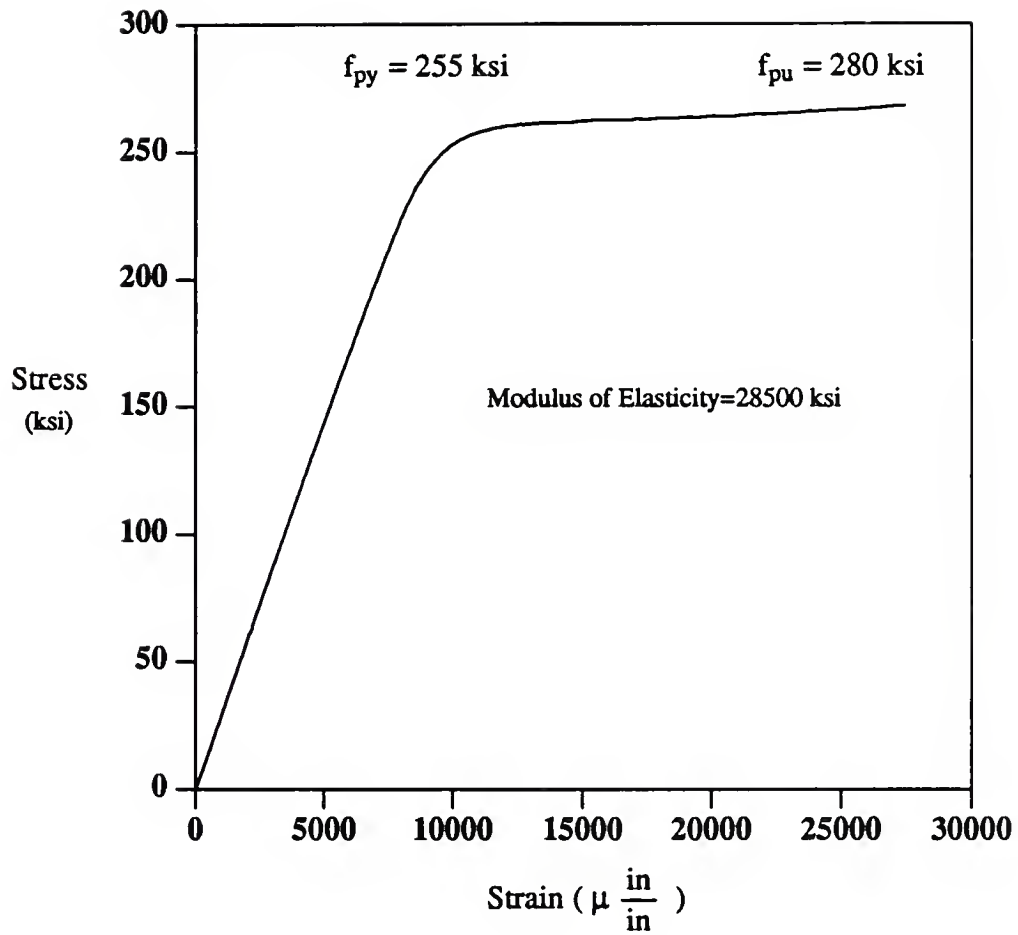


Figure 3.11 Measured Stress-Strain Behavior
of Prestressing Strands (Stress Relieved)
Specimen Set 1

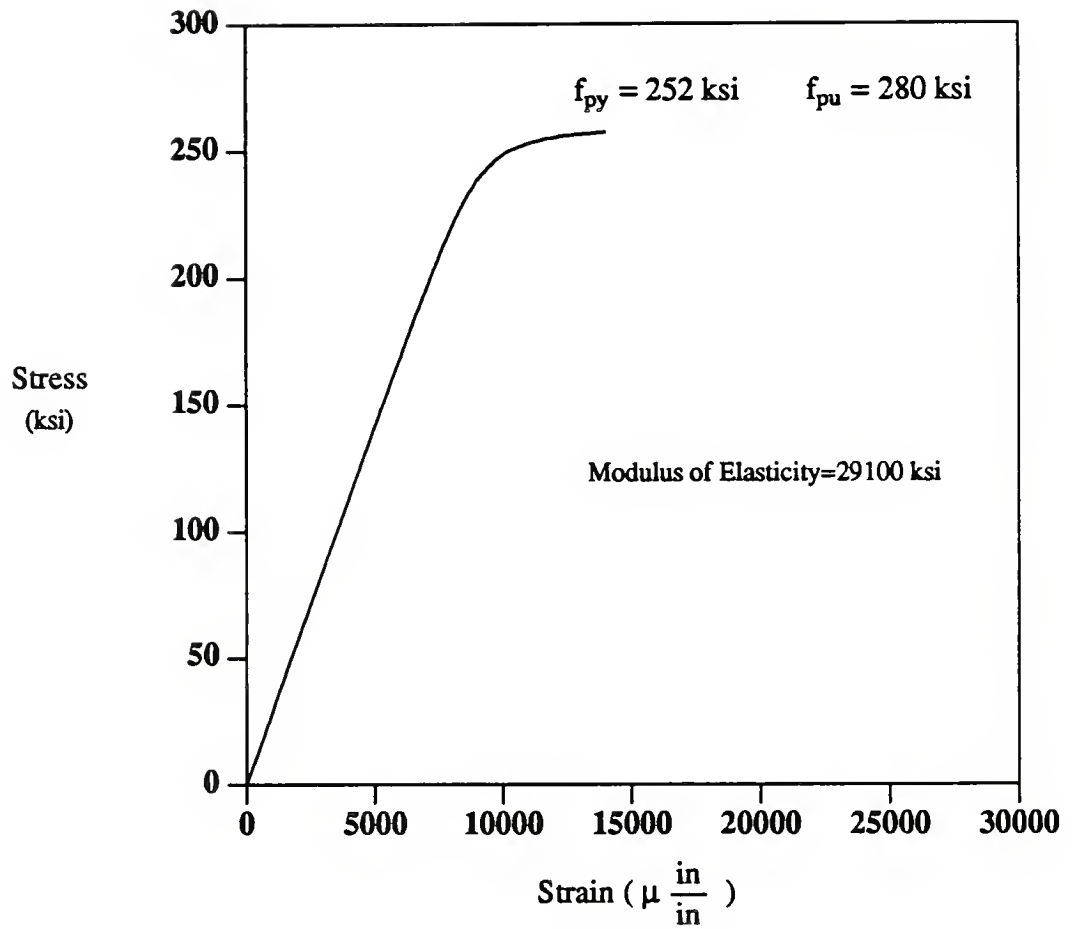


Figure 3.12 Measured Stress-Strain Behavior
of Prestressing Strands (Low-Lax)
Specimen Sets 2 and 3

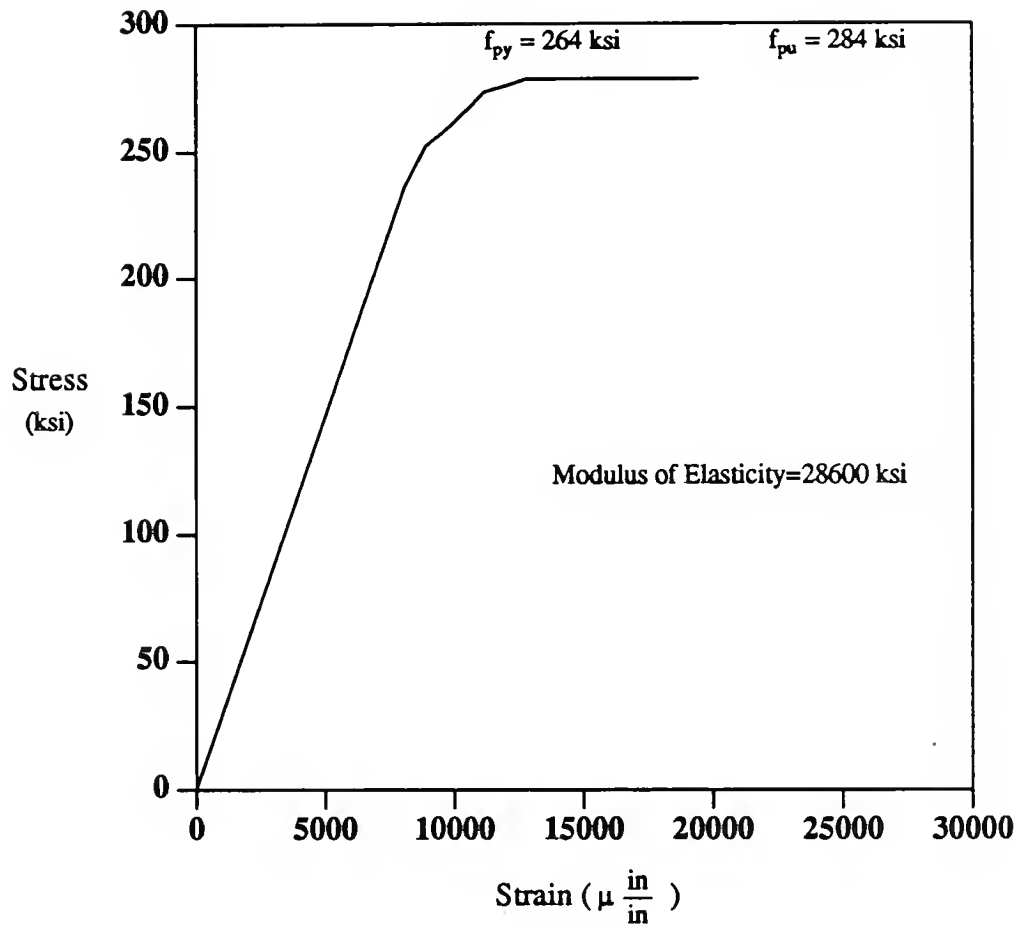


Figure 3.13 Measured Stress-Strain Behavior
of Prestressing Strands (Low-Lax)
(Specimen Sets 4 and 5)

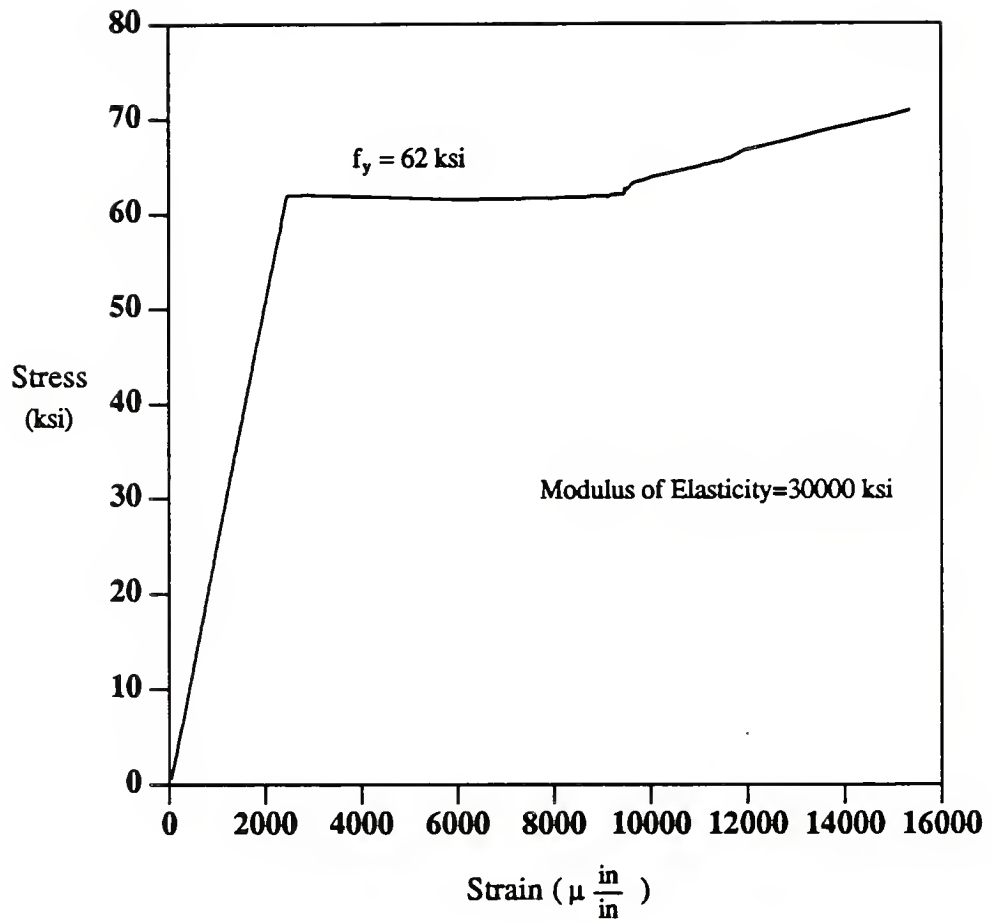


Figure 3.14 Measured Stress-Strain Behavior of Mild Steel
#6 Bar, Grade 60 (Specimen Set 1)

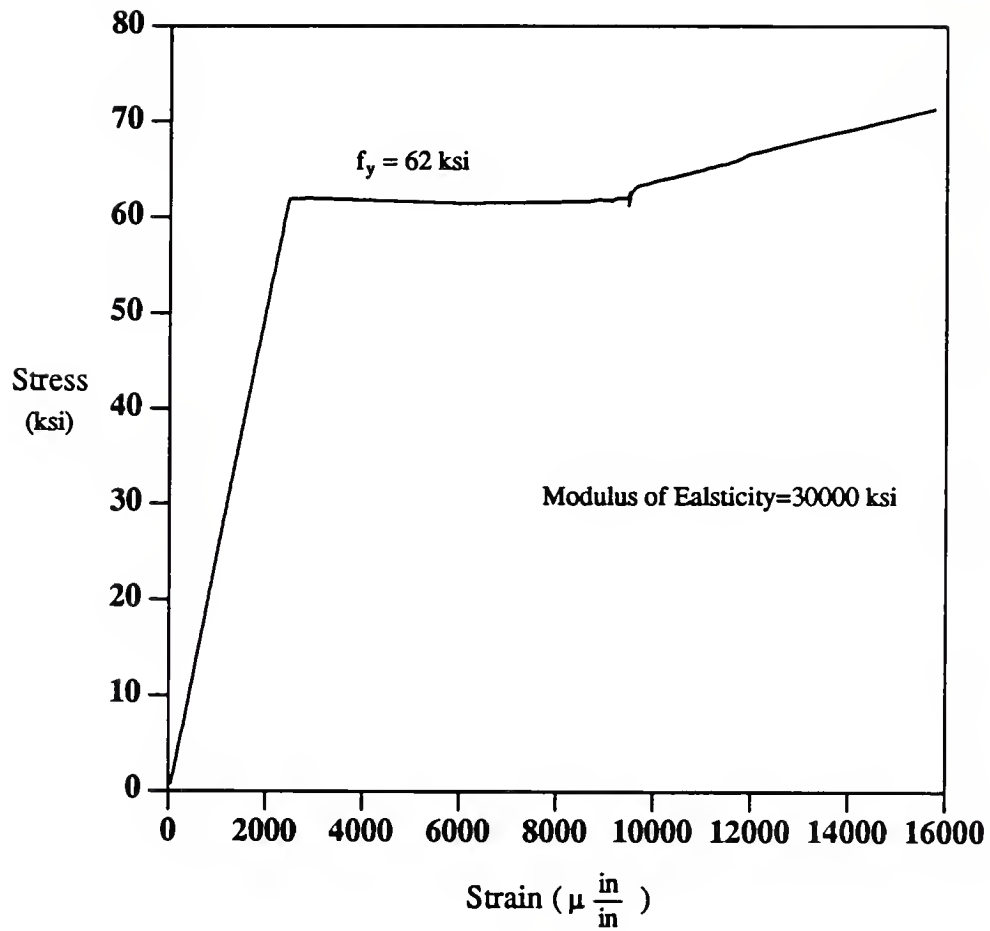


Figure 3.15 Measured Stress-Strain Behavior of Mild Steel
#6 Bar, Grade 60 (Specimen Sets 2 and 3)

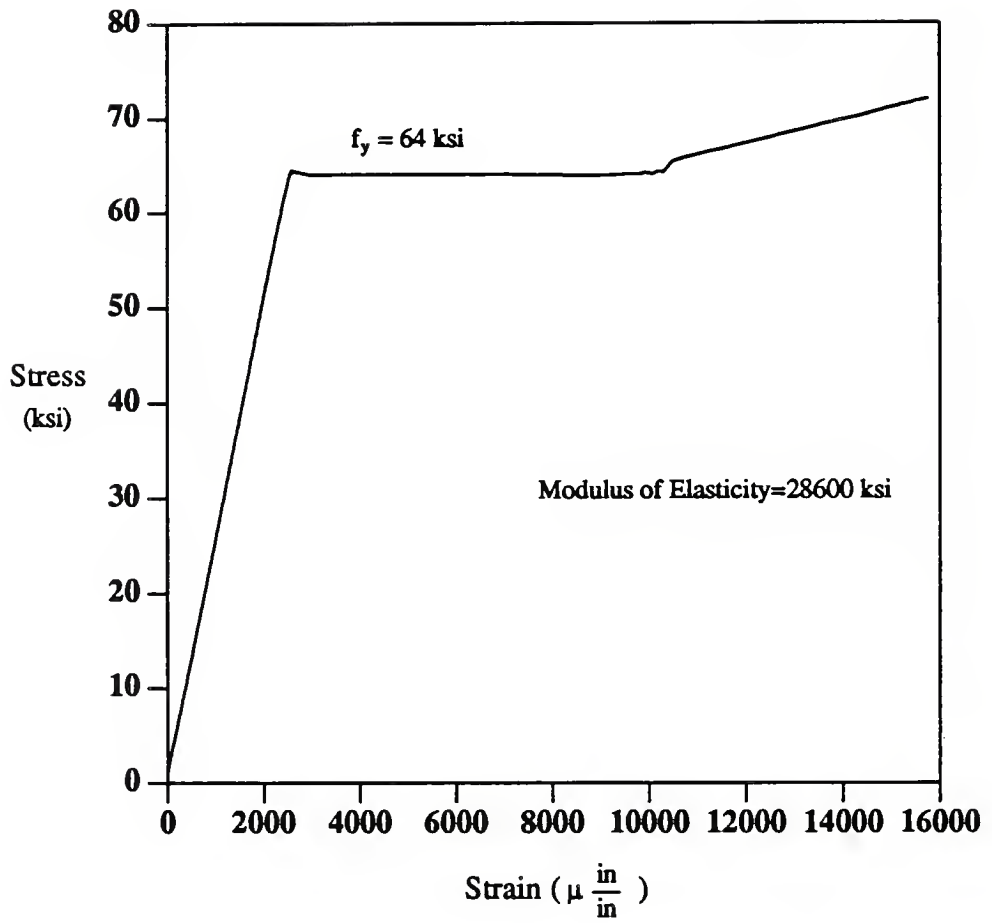


Figure 3.16 Measured Stress-Strain Behavior of Mild Steel
#6 Bar, Grade 60 (Specimen Set 4)

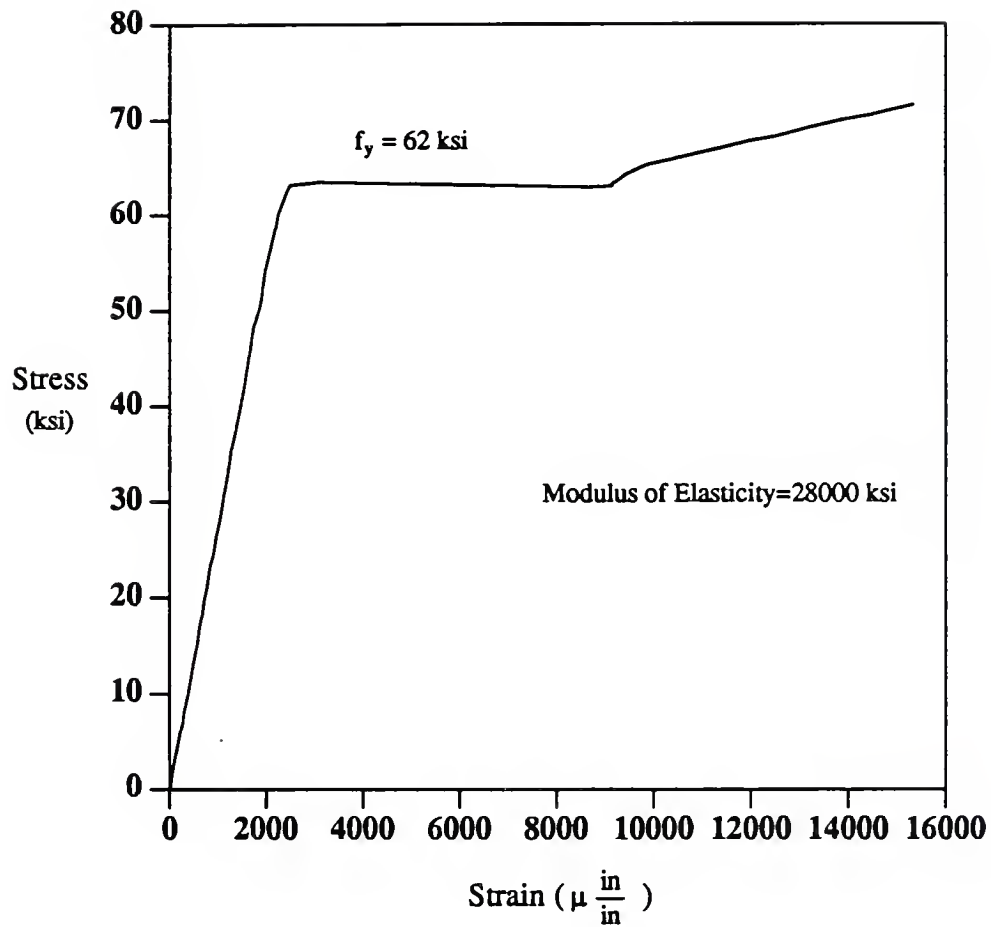


Figure 3.17 Measured Stress-Strain Behavior of Mild Steel
#6 Bar, Grade 60 (Specimen Set 5)

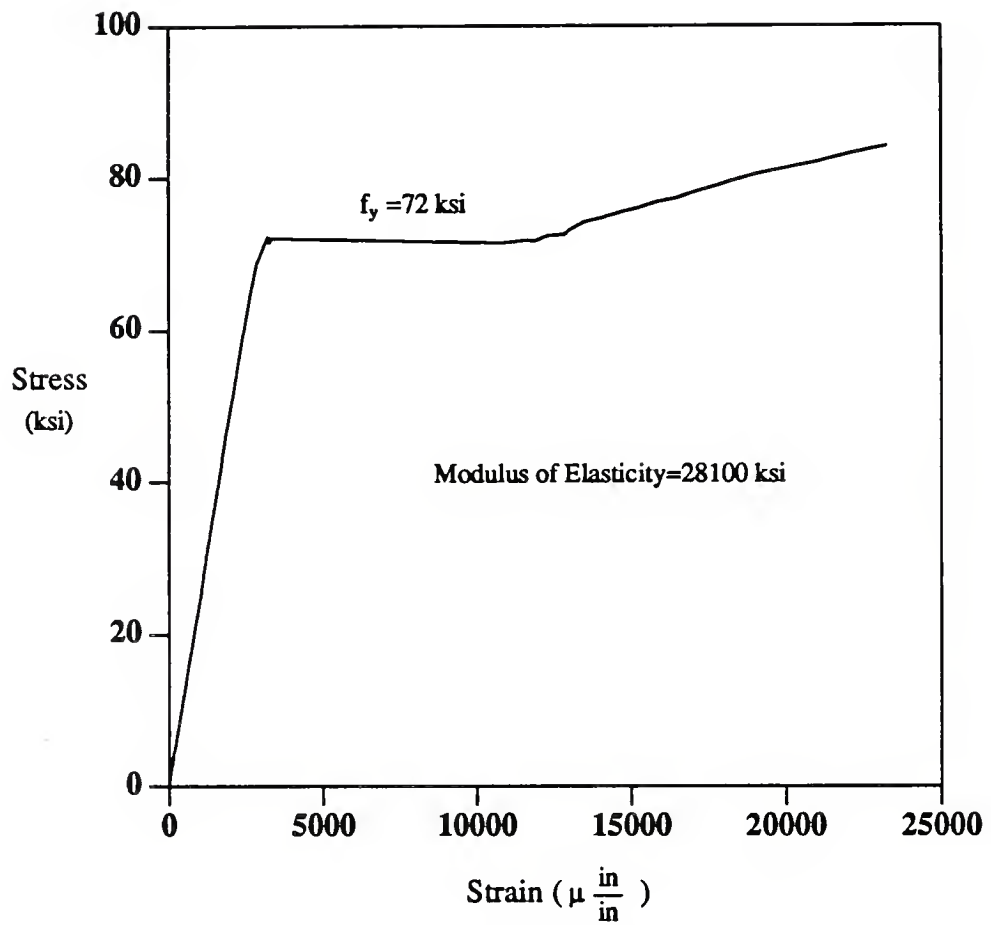


Figure 3.18 Measured Stress-Strain Behavior of Mild Steel
#3 Bar, Grade 60 (Specimen Set 1)

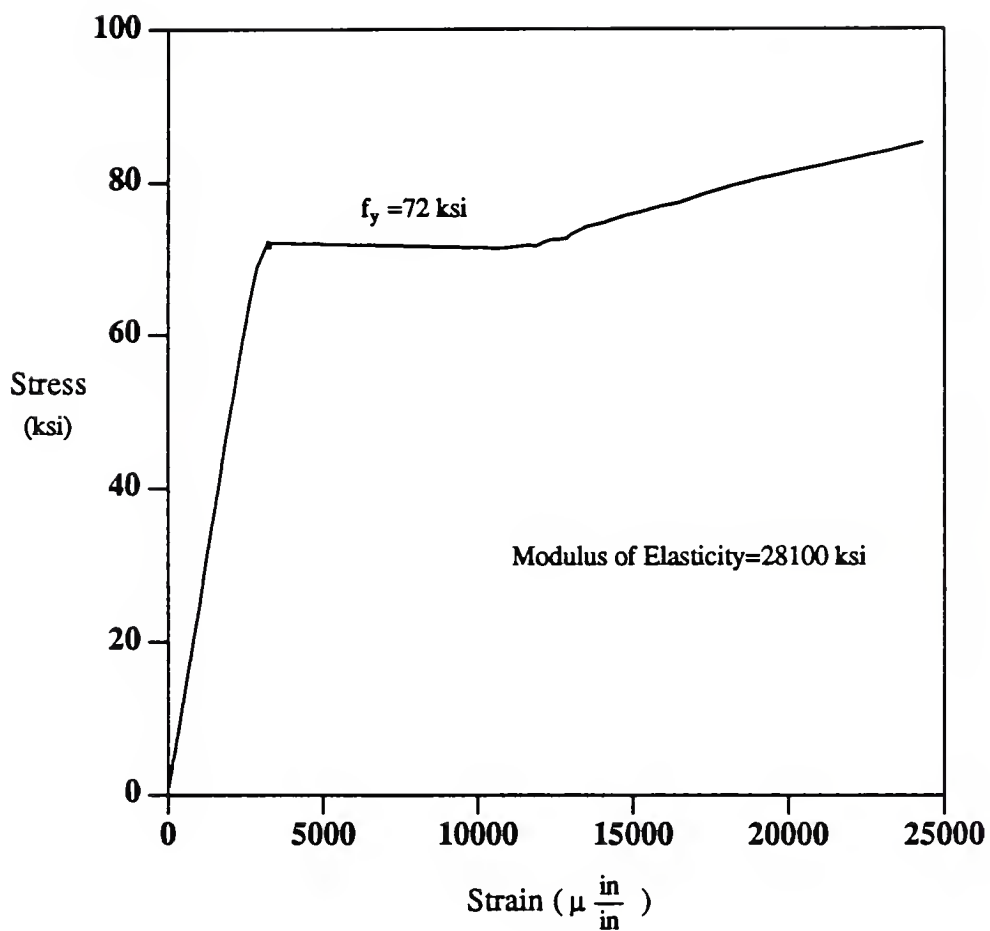


Figure 3.19 Measured Stress-Strain Behavior of Mild Steel
#3 Bar, Grade 60 (Specimen Sets 2 and 3)

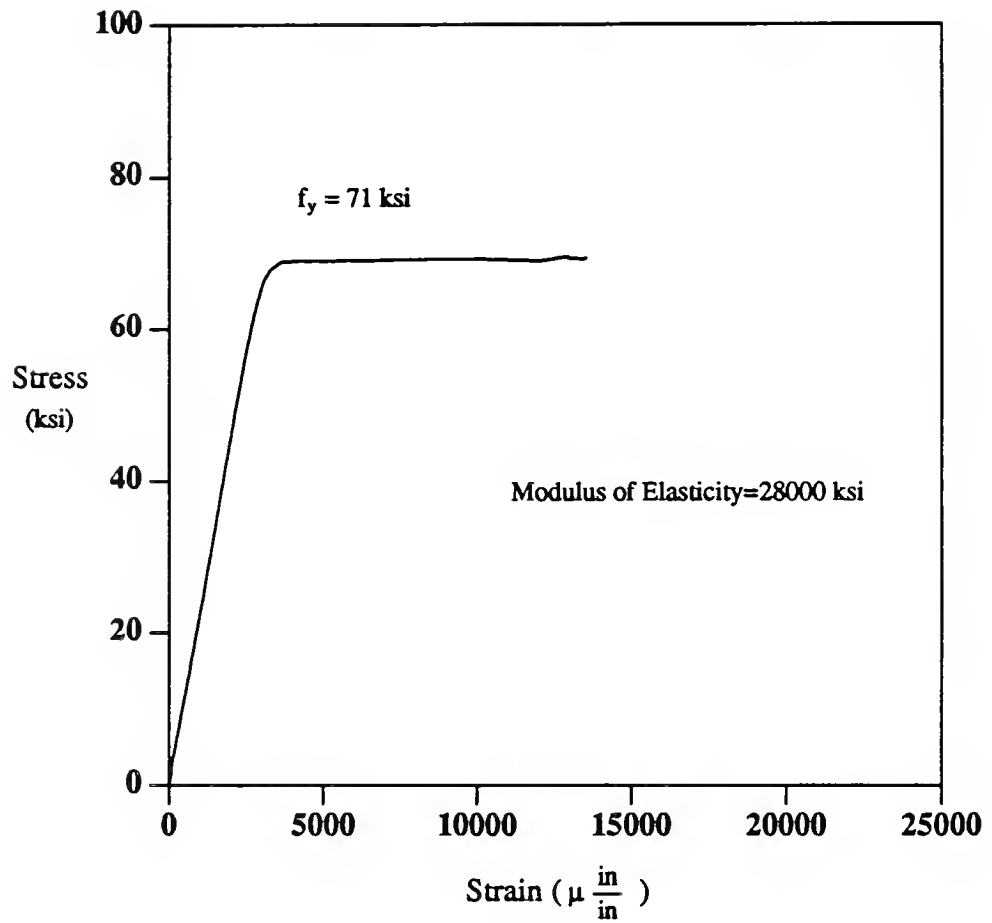


Figure 3.20 Measured Stress-Strain Behavior of Mild Steel
#3 Bar, Grade 60 (Specimen Sets 4 and 5)

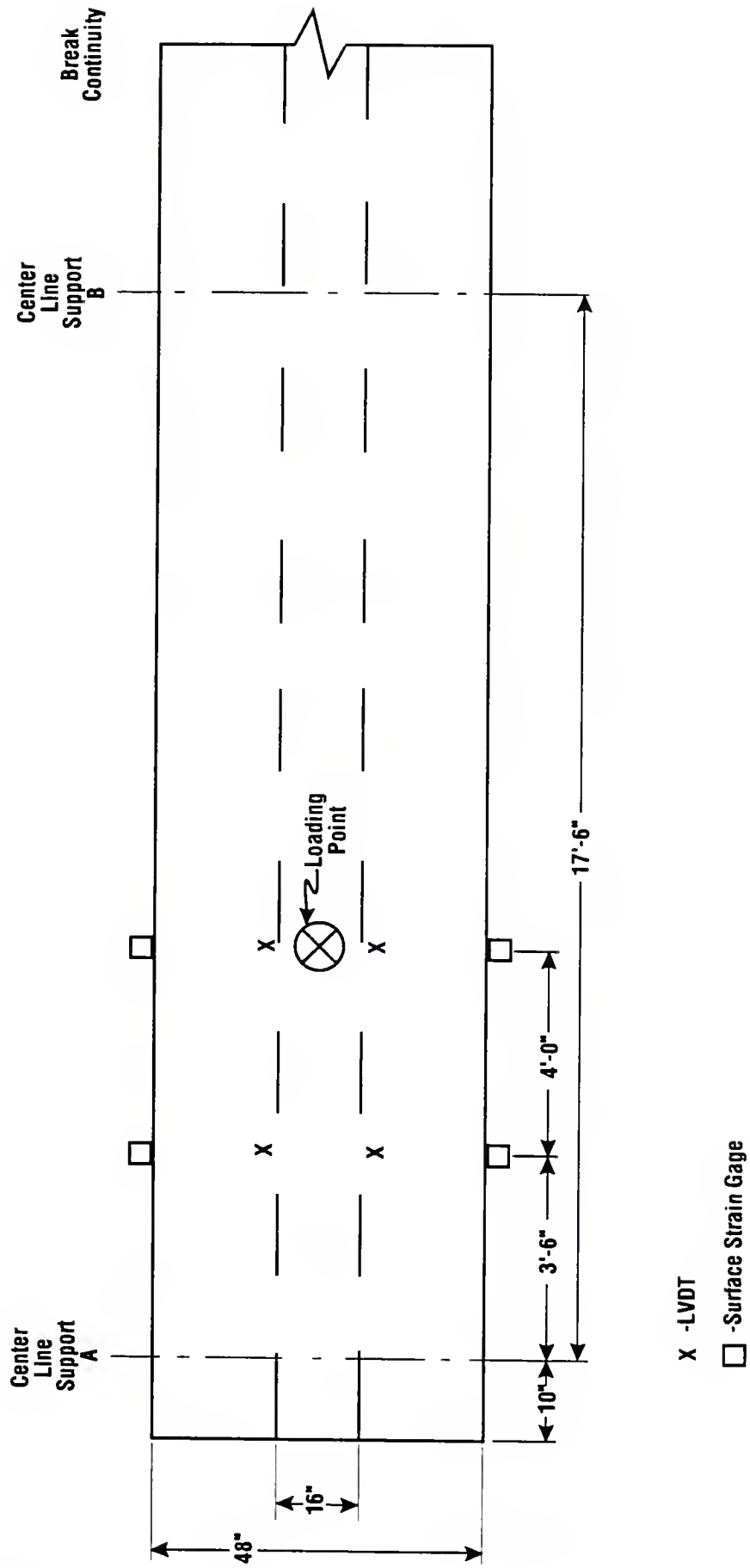


Figure (3.21) Top View of Location of LVDT's and Surface Strain Gages for Specimen Sets 1, 2, and 3.

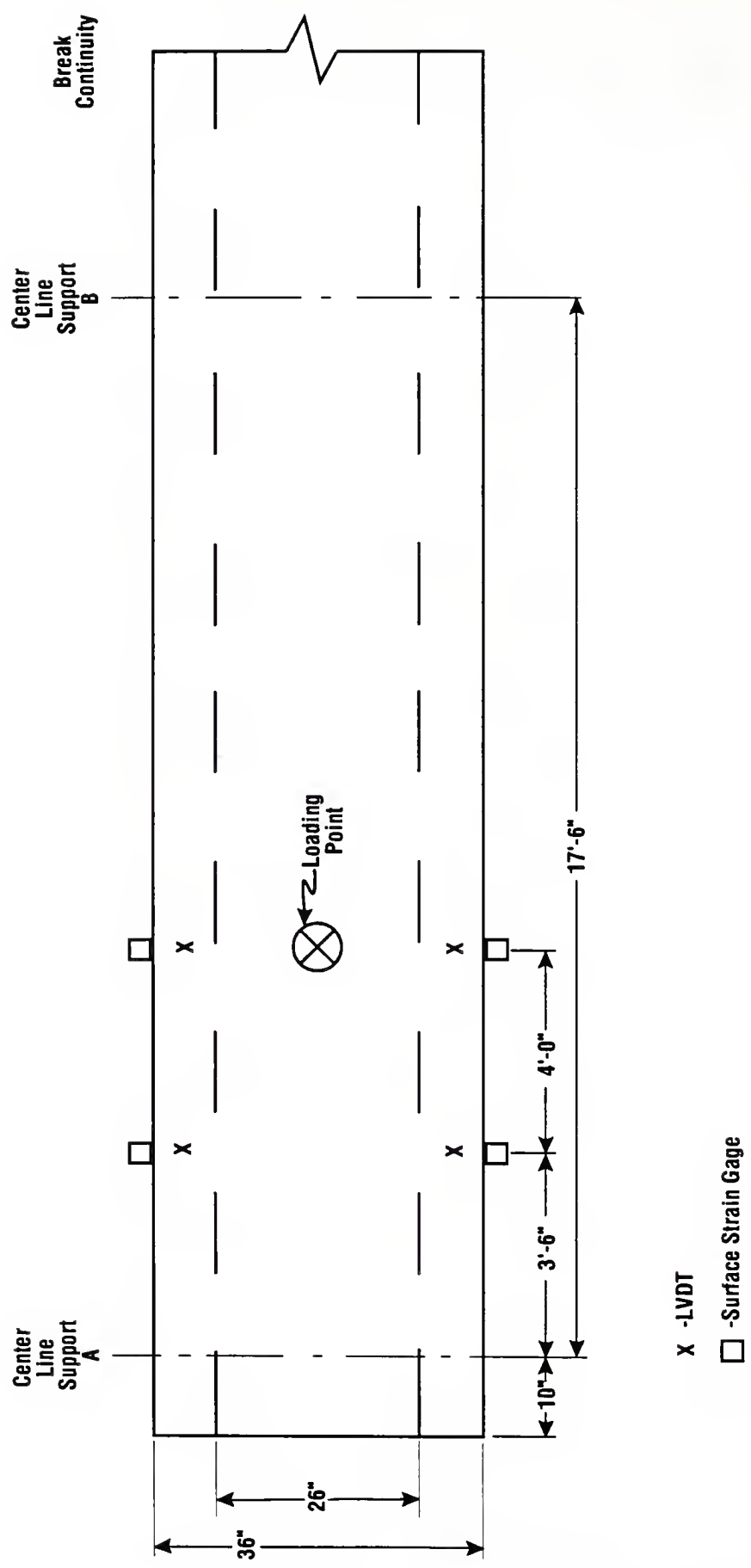


Figure (3.22) Top View of Location of LVDT's and Surface Strain Gages for Specimen Set 4.

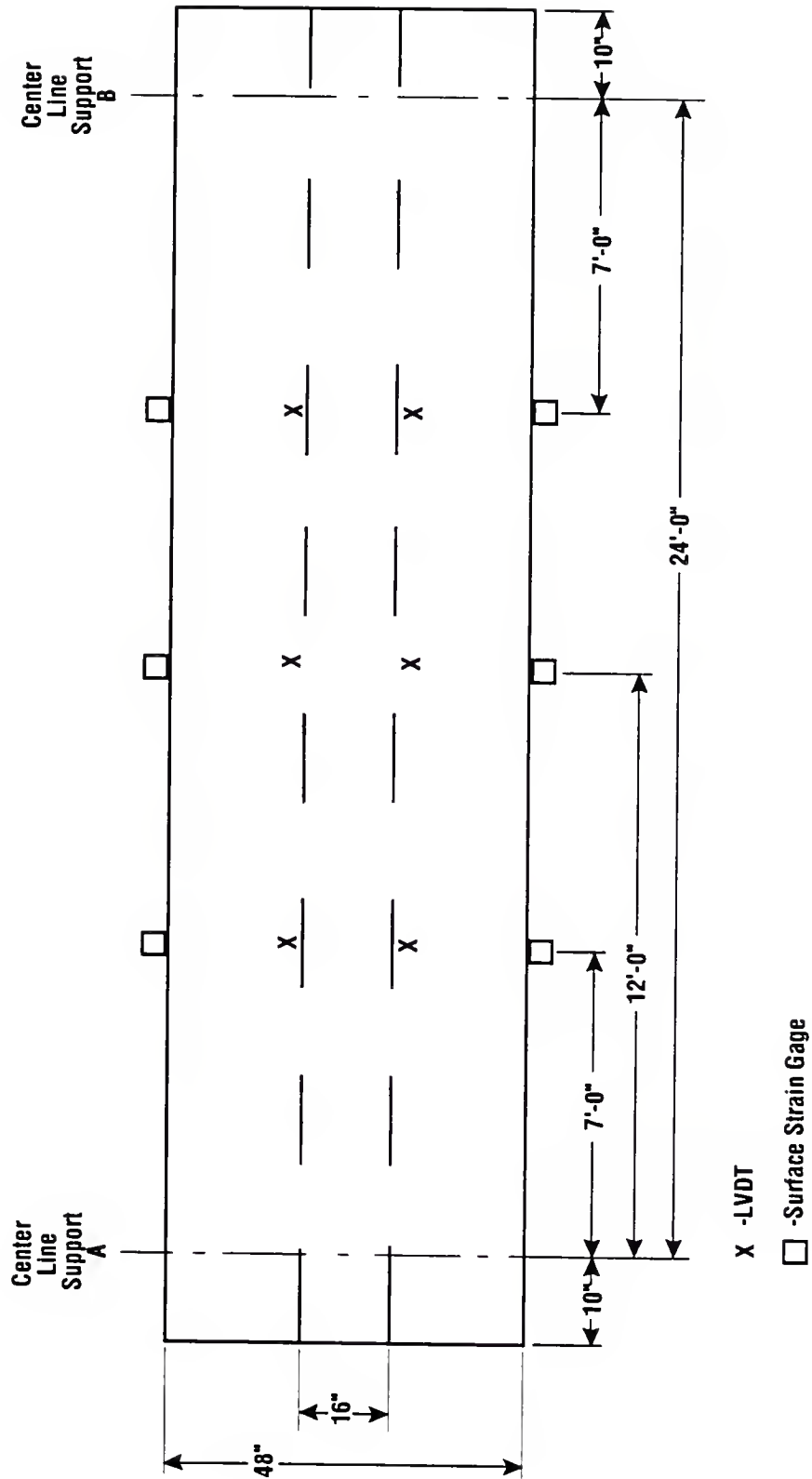


Figure (3.23) Top View of Location of LVDT's and Surface Strain Gages for Specimen Set 5.

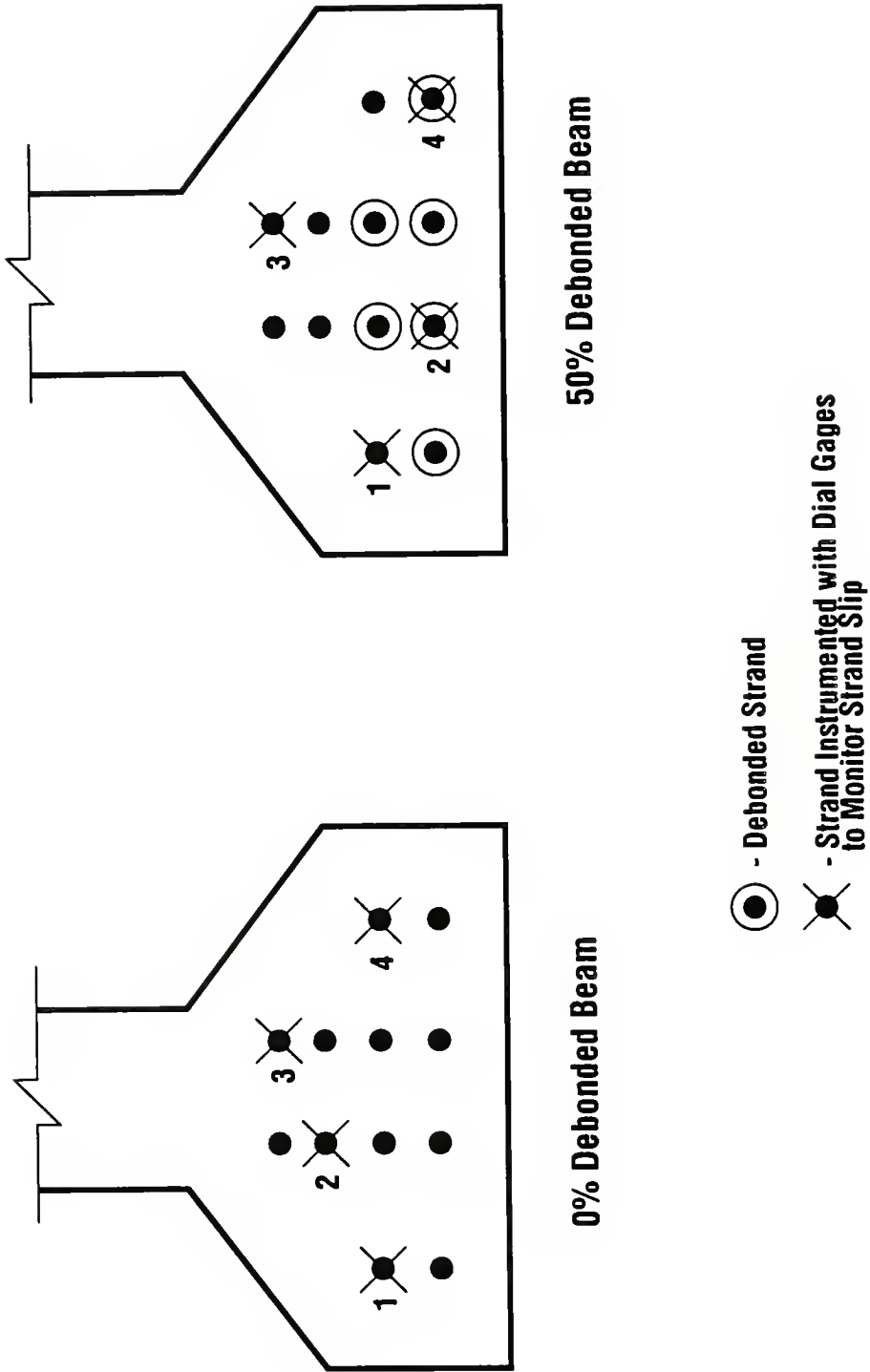


Figure (3.24) Strand Slip Instrumentation for Specimen Set 1

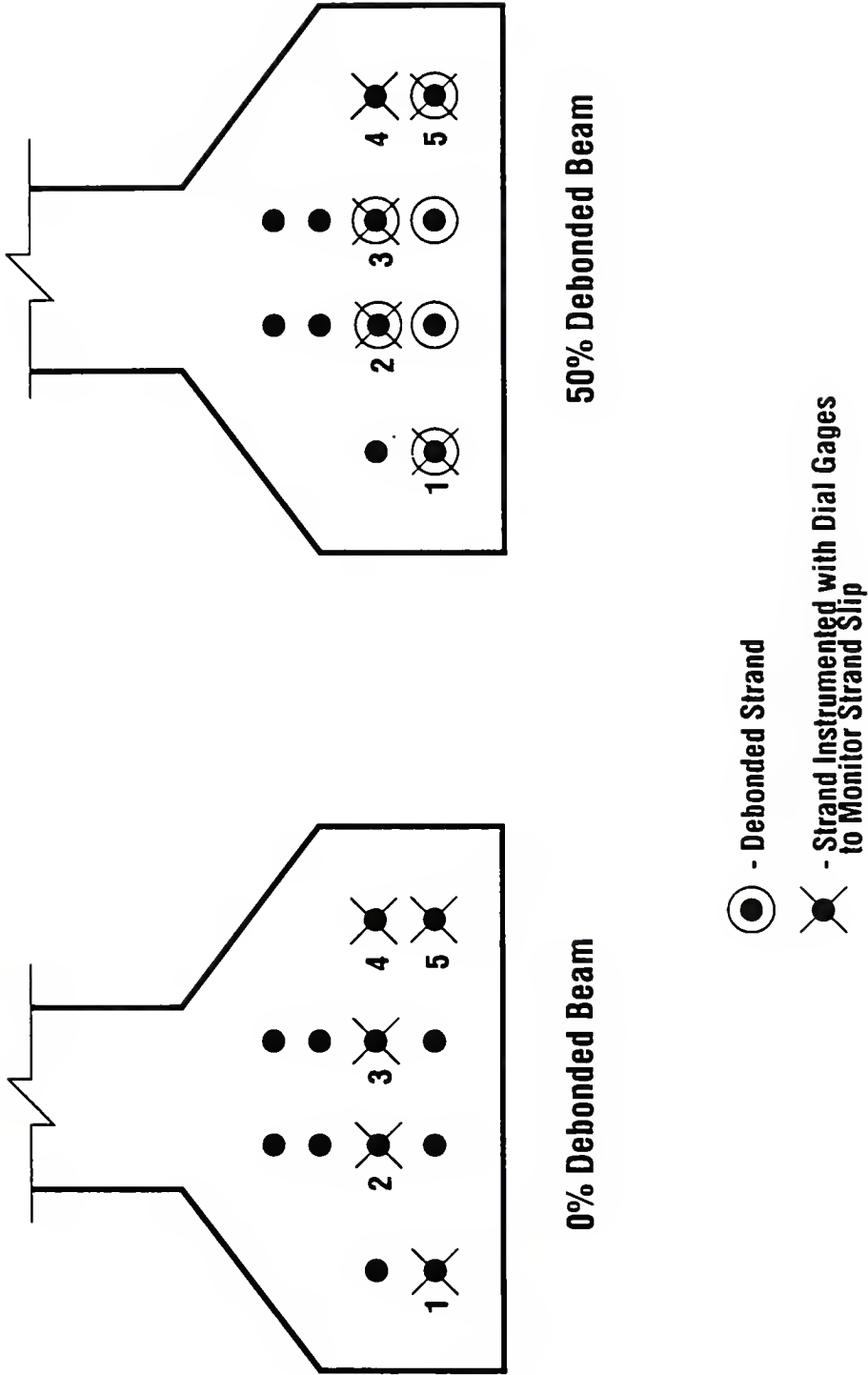


Figure (3.25) Strand Slip Instrumentation for Specimen Set 2.

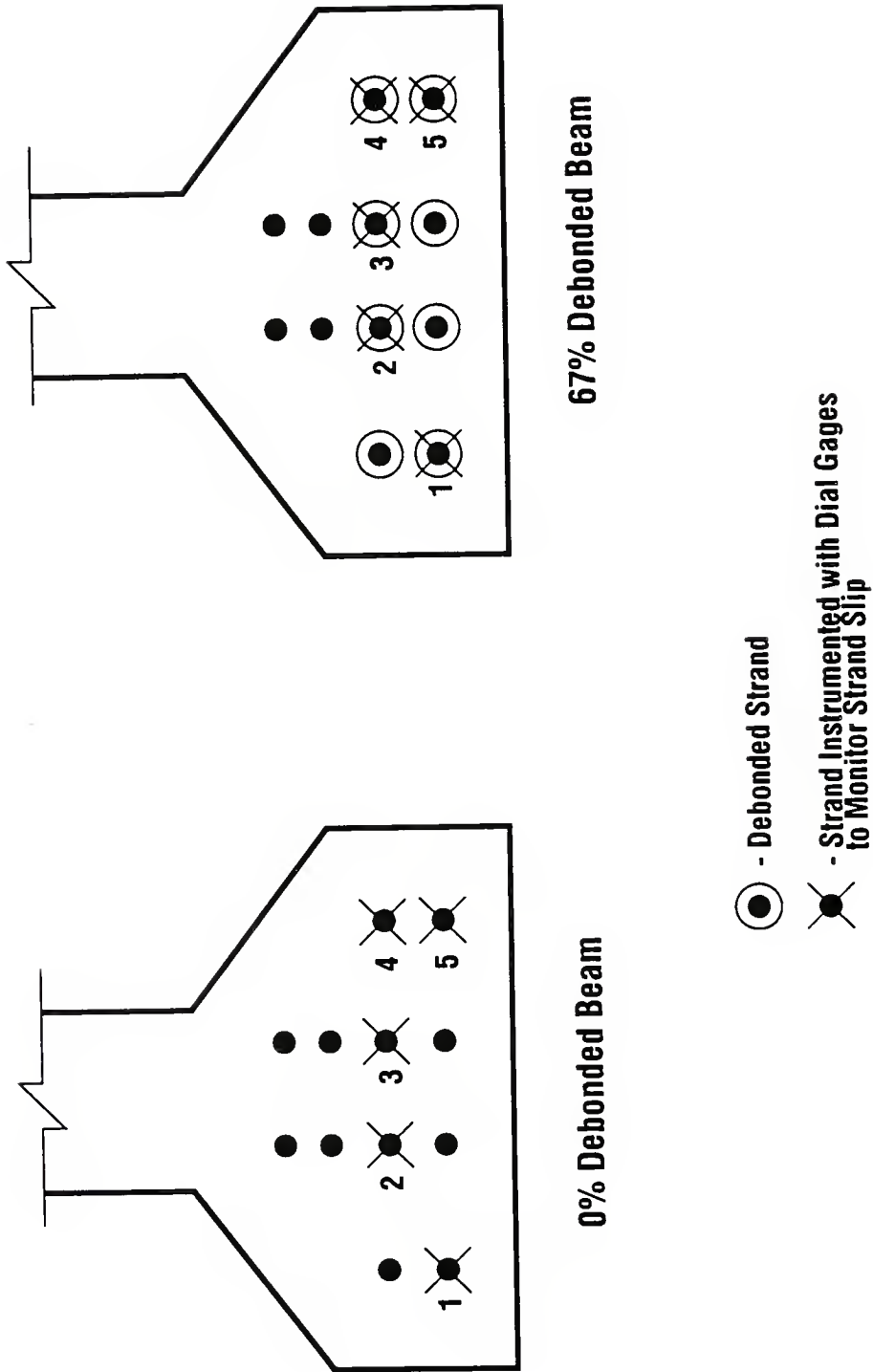


Figure (3.26) Strand Slip Instrumentation for Specimen Set 3.

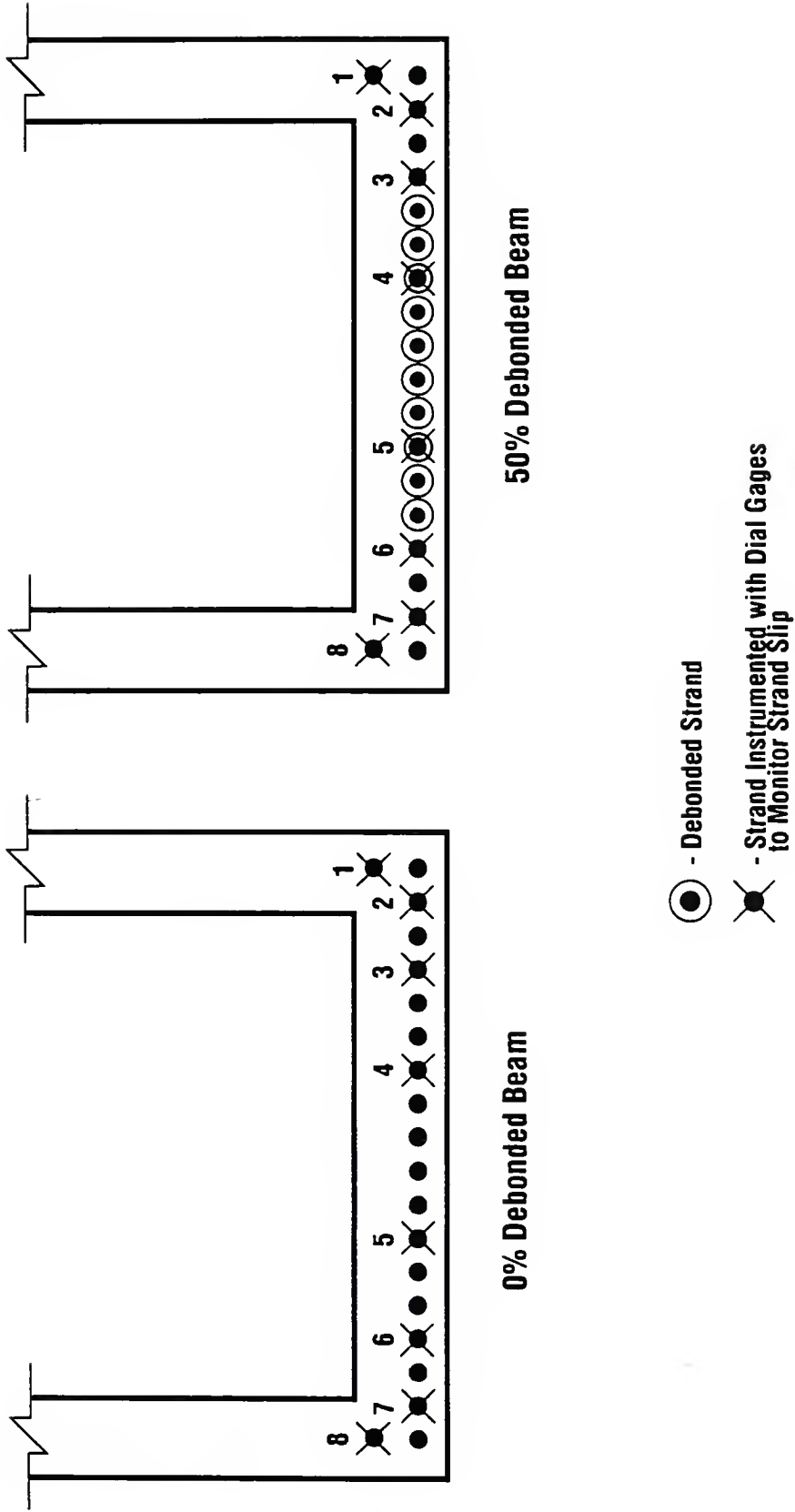


Figure (3.27) Strand Slip Instrumentation for Specimen Set 4.

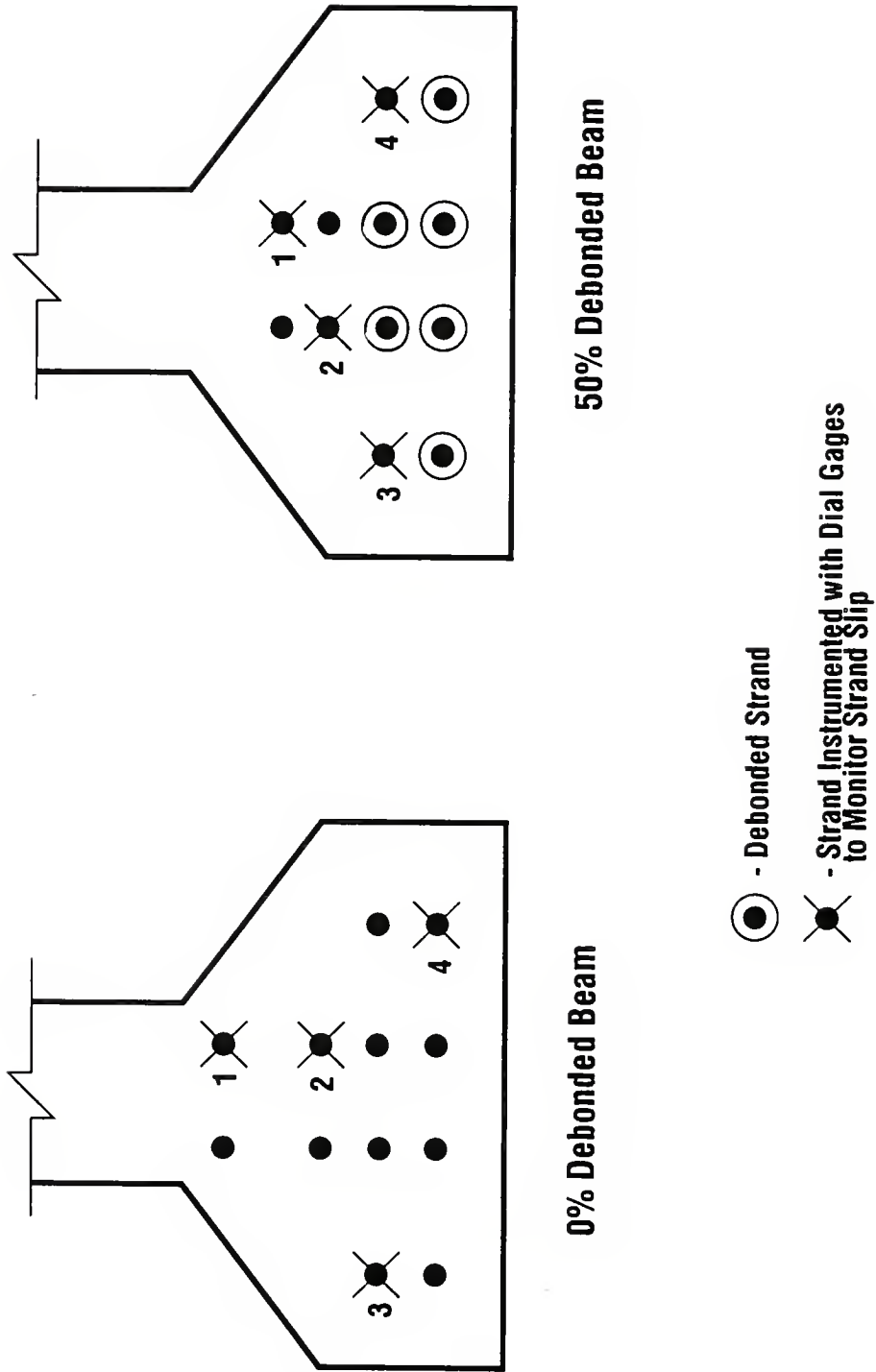
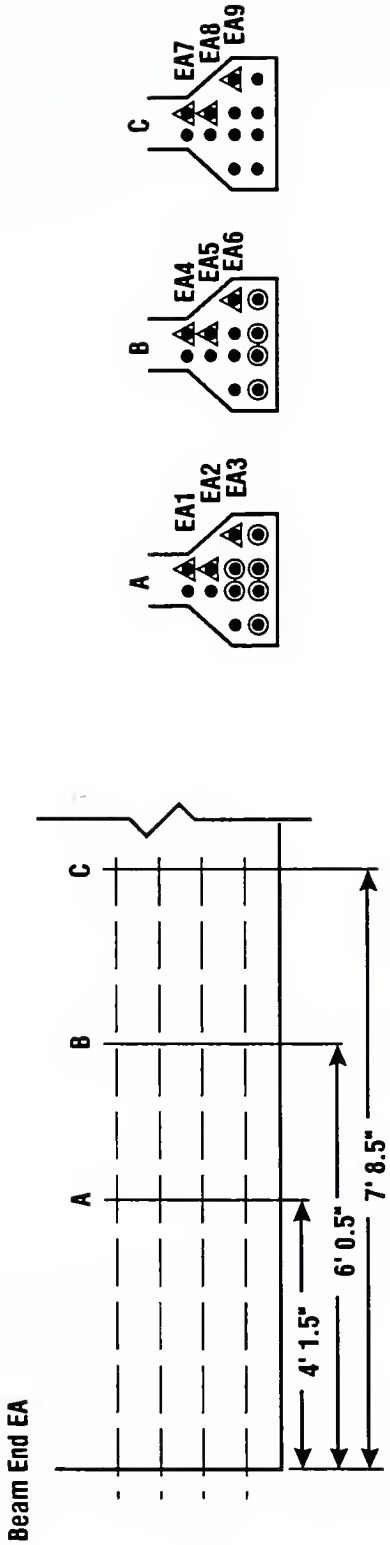


Figure (3.28) Strand Slip Instrumentation for Specimen Set 5.

Beam With 50% Debonding



Beam With 0% Debonding

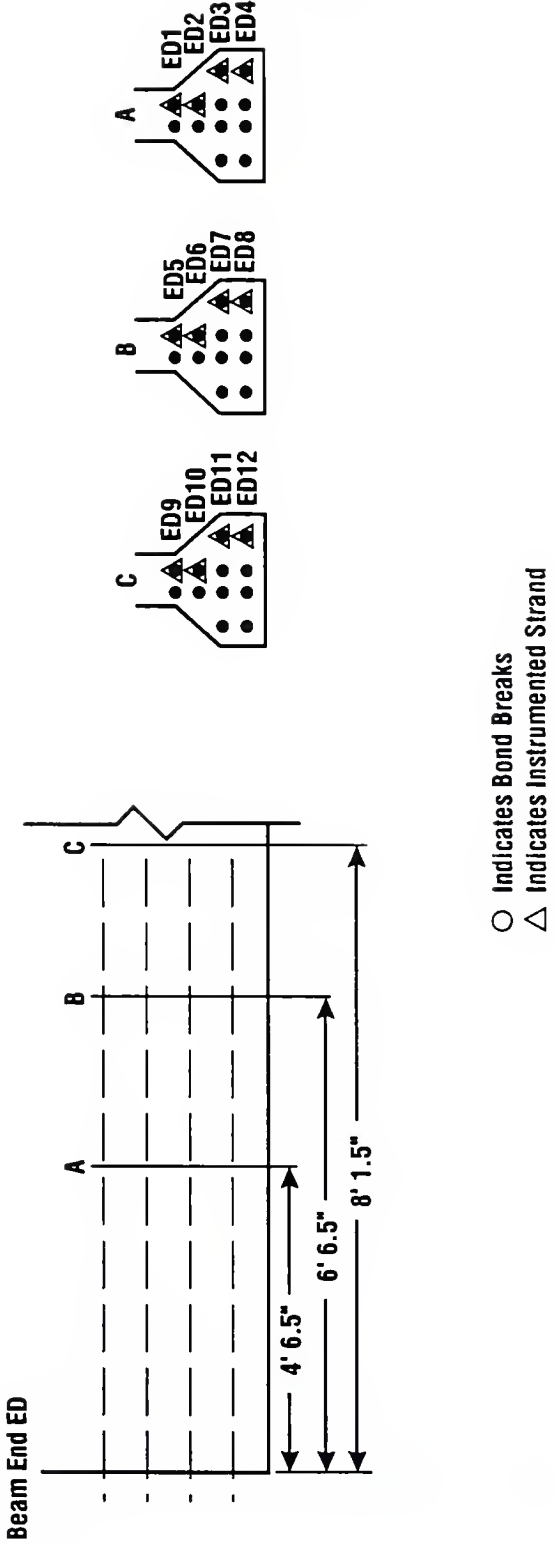


Figure (3.29) Strand Debonding Scheme and Instrumentation for Specimen Set 1.



Figure 3.30 Crack Pattern of 0% Debonded Beam Before Slab Failure
Specimen Set 1

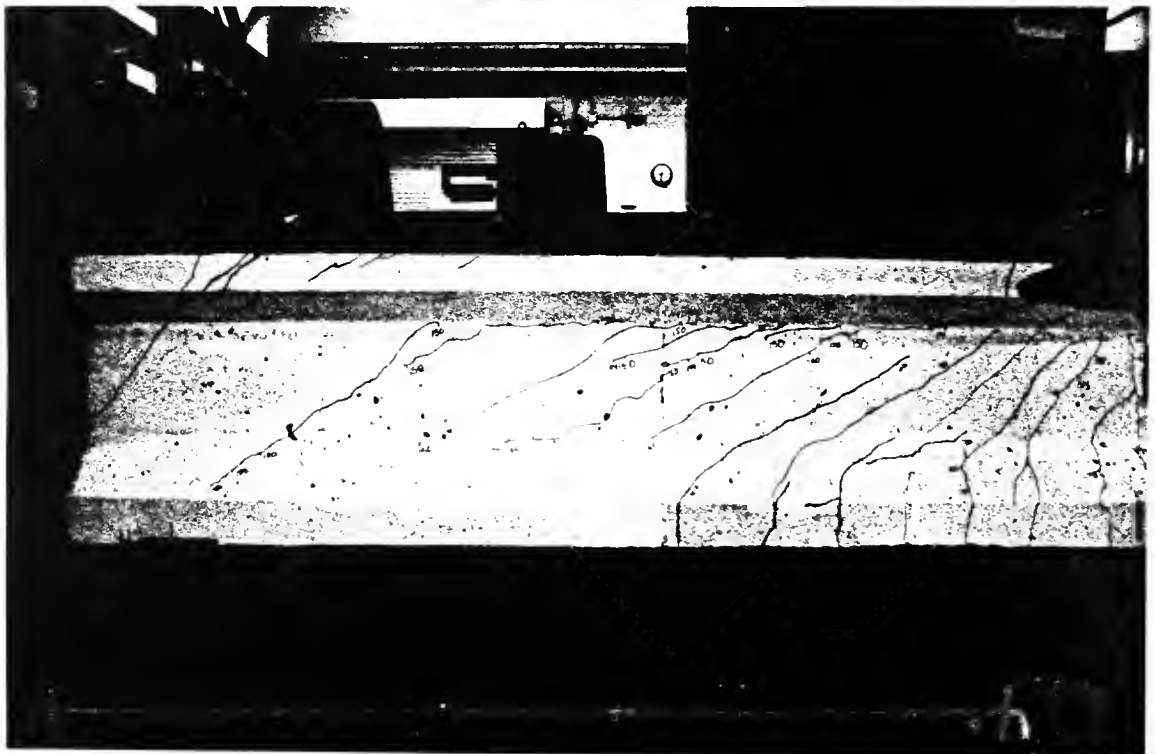


Figure 3.31 Crack Pattern of 0% Debonded Beam at Failure
Specimen Set 1

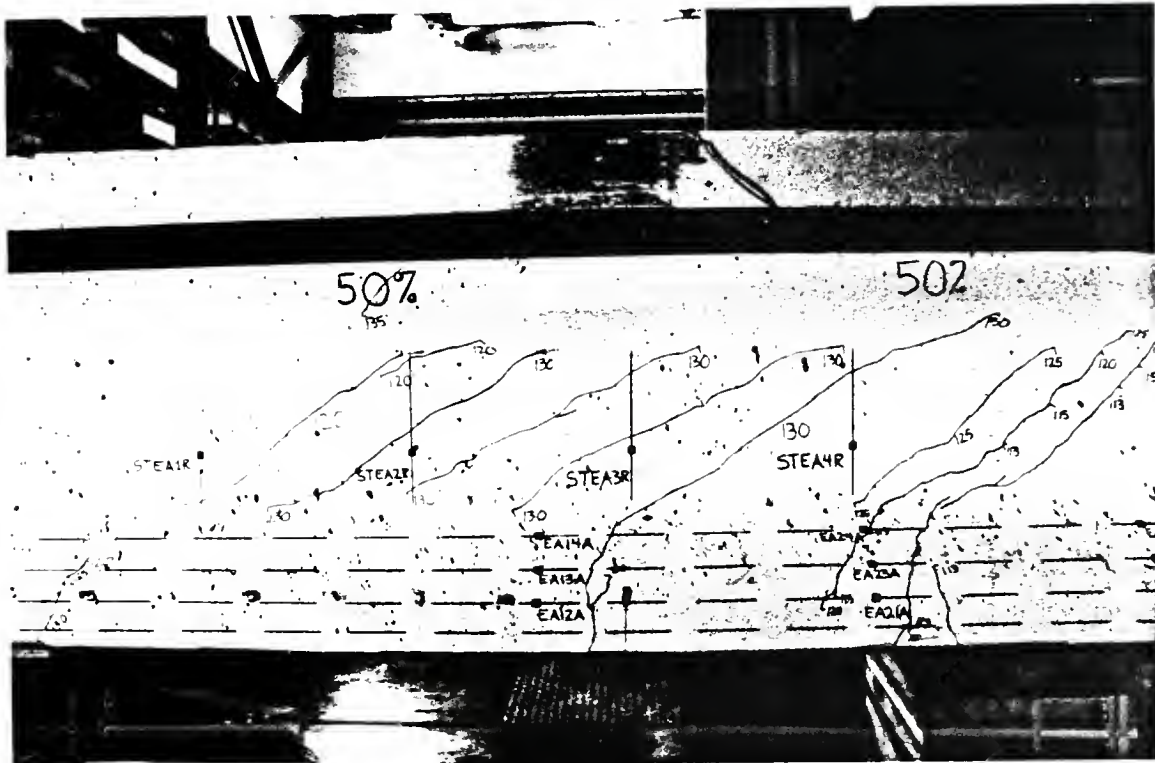


Figure 3.32 Crack Pattern of 50% Debonded Beam Before Slab Failure
Specimen Set 1



Figure 3.33 Crack Pattern of 50% Debonded Beam at Failure
Specimen Set 1

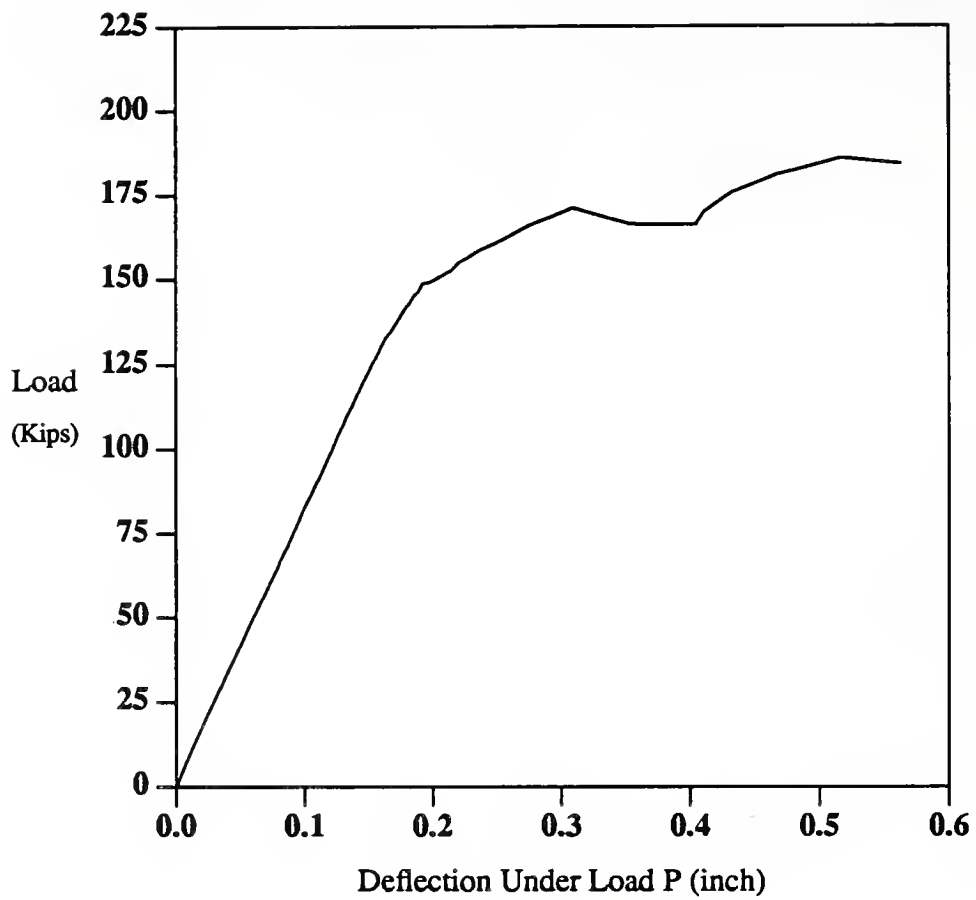


Figure 3.34 Load-Deflection Relationship, Beam with 0% Debonding
(Specimen Set 1)

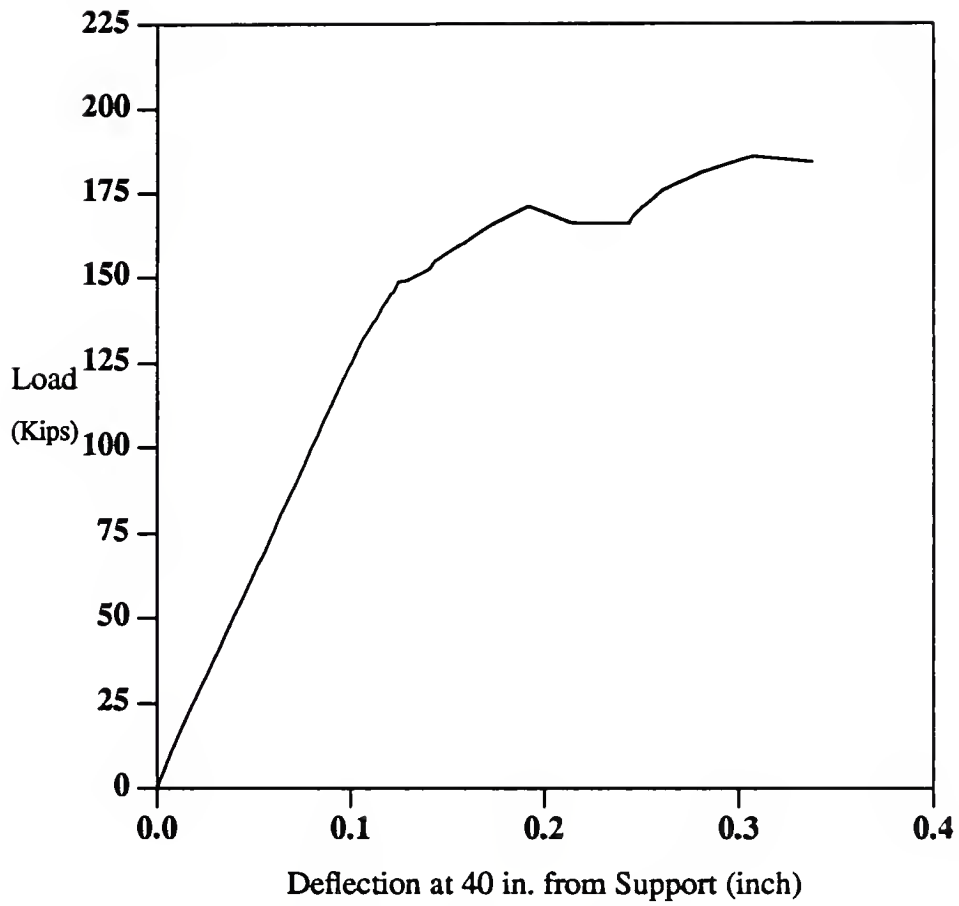


Figure 3.35 Load-Deflection Relationship, Beam with 0% Debonding
(Specimen Set 1)

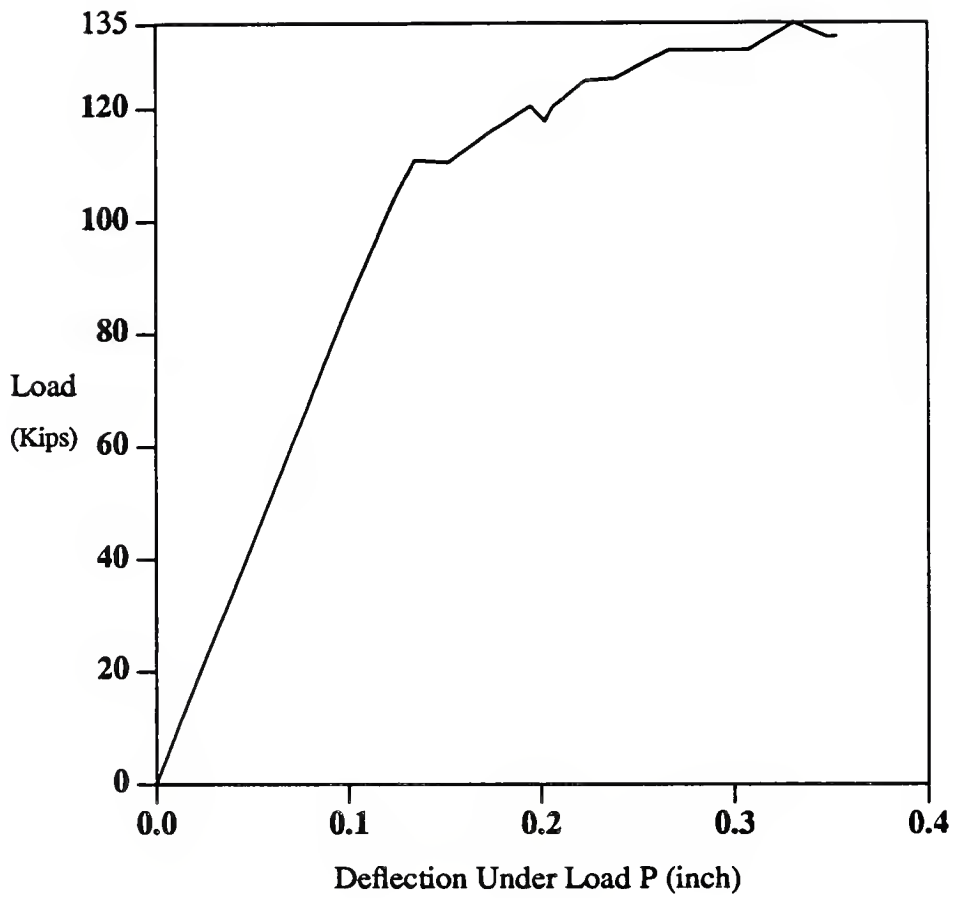


Figure 3.36 Load-Deflection Relationship, for Simply Supported Beam
Beam with 50% Debonding (Specimen Set 1)

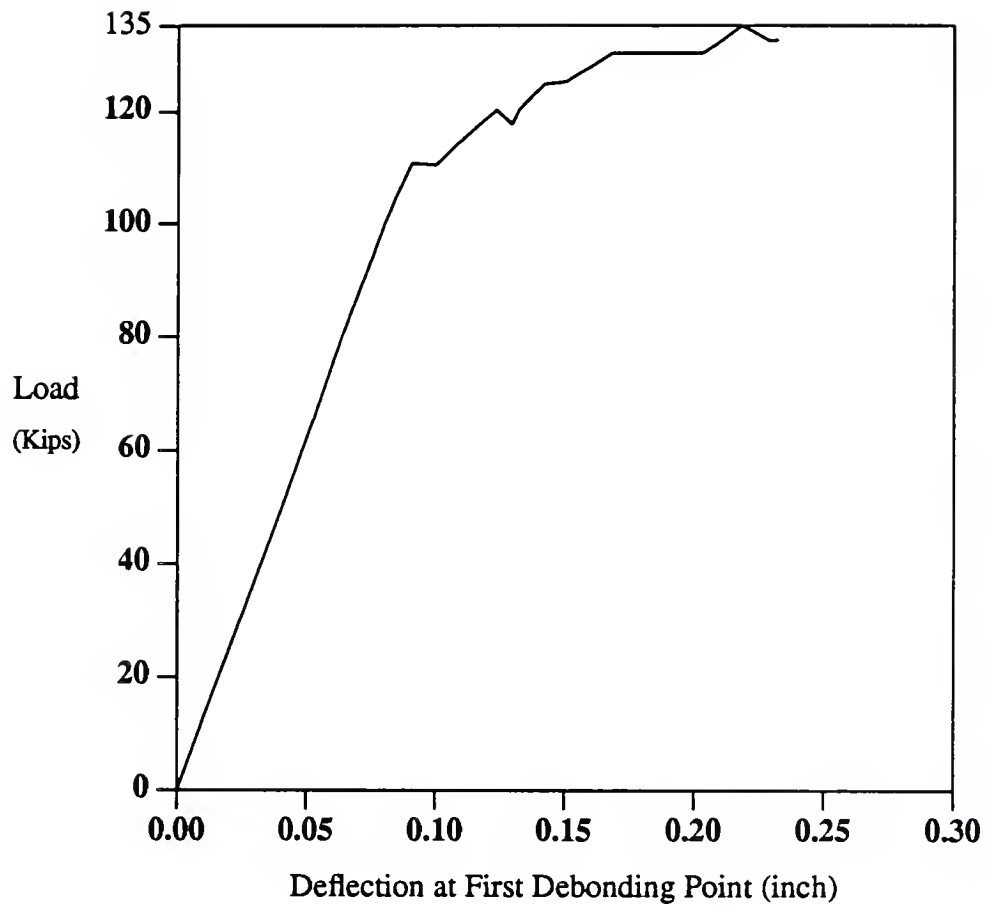


Figure 3.37 Load-Deflection Relationship, for Simply Supported Beam
Beam with 50% Debonding (Specimen Set 1)

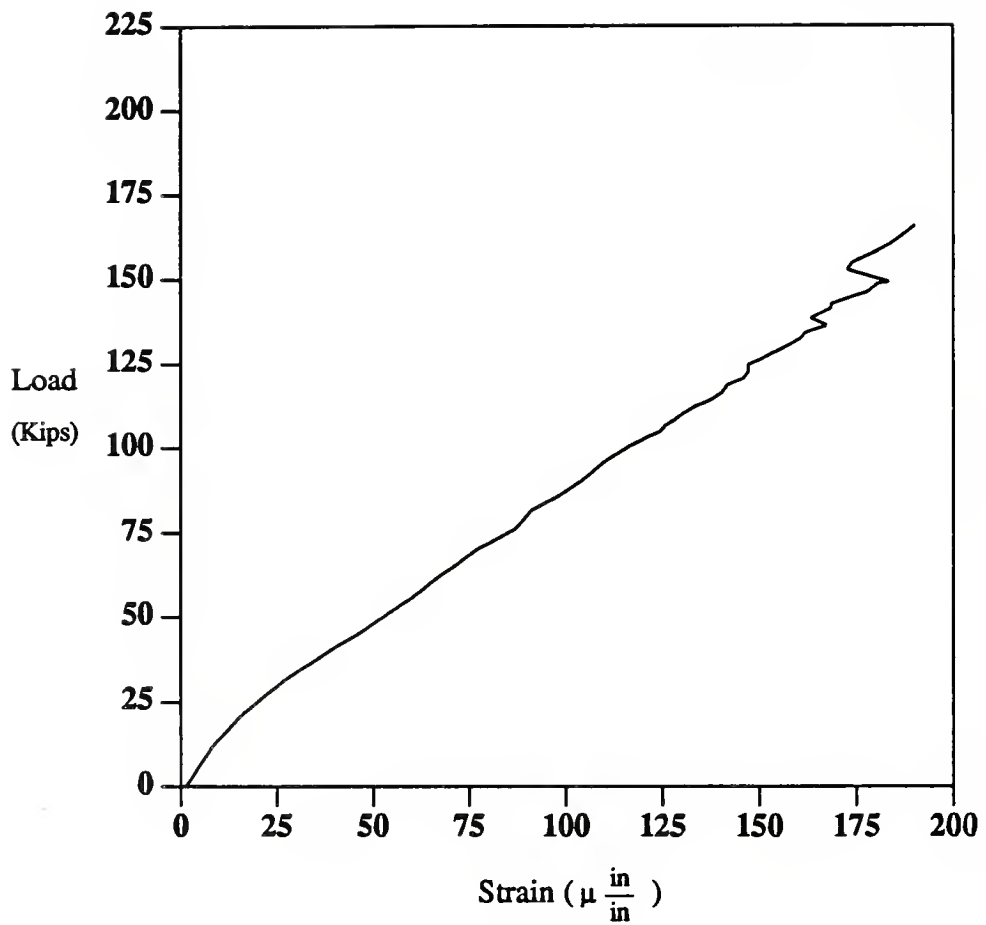


Figure 3.38 Surface Strain in Slab, 40 in. from the Support Beam with 0% Debonding (Specimen Set 1)

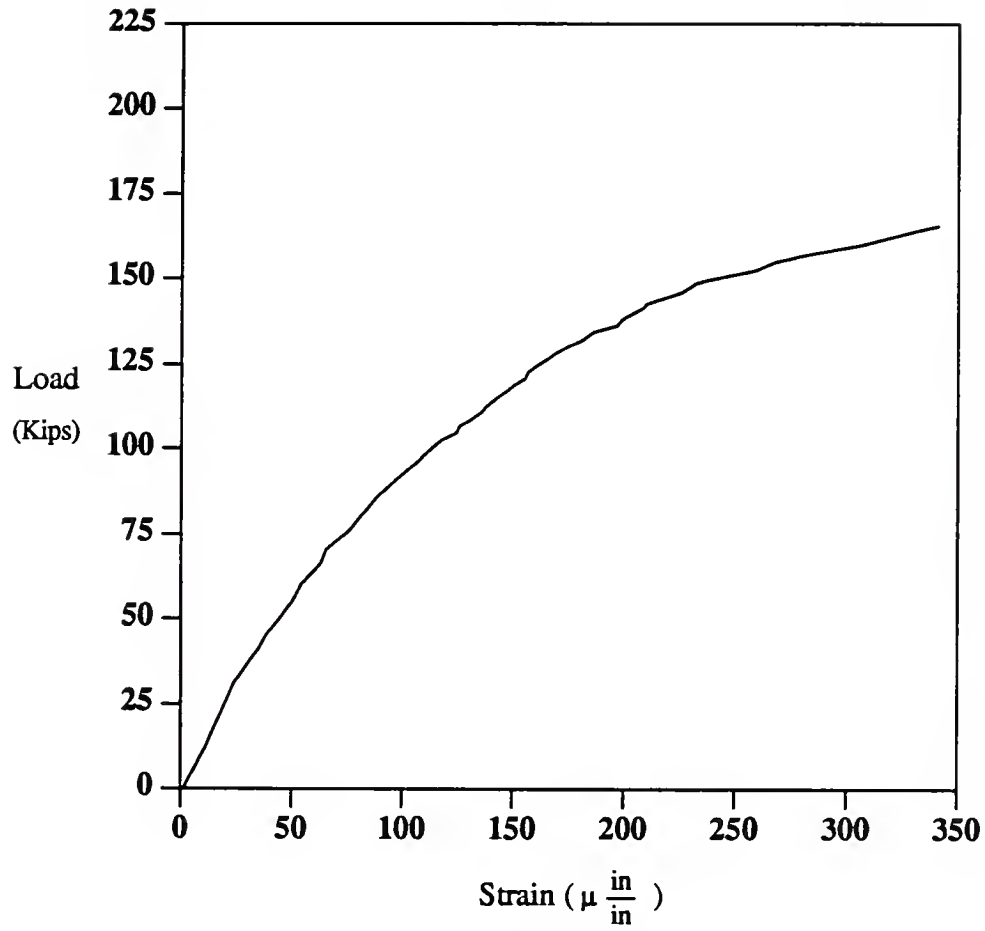


Figure 3.39 Surface Strain in Slab, at the Point Load
Beam with 0% Debonding (Specimen Set 1)

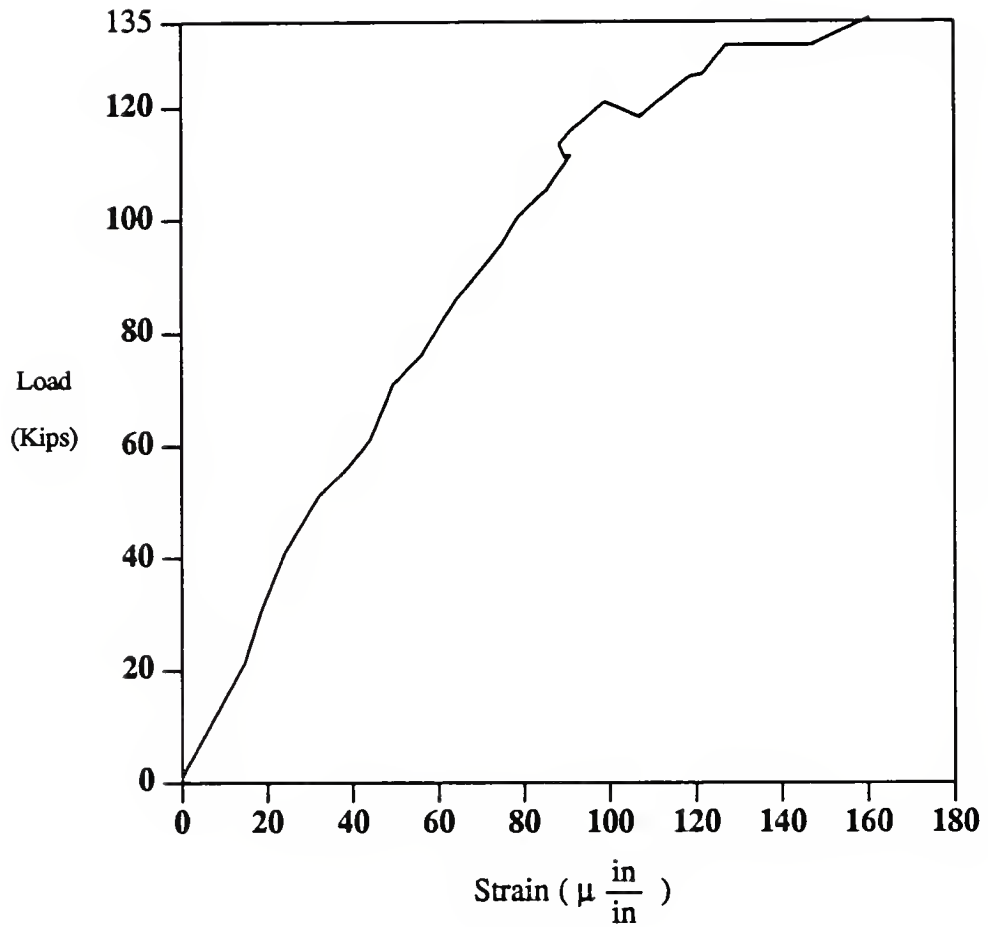


Figure 3.40 Surface Strain in Slab, at First Debonding Point
Beam with 50% Debonding (Specimen Set 1)

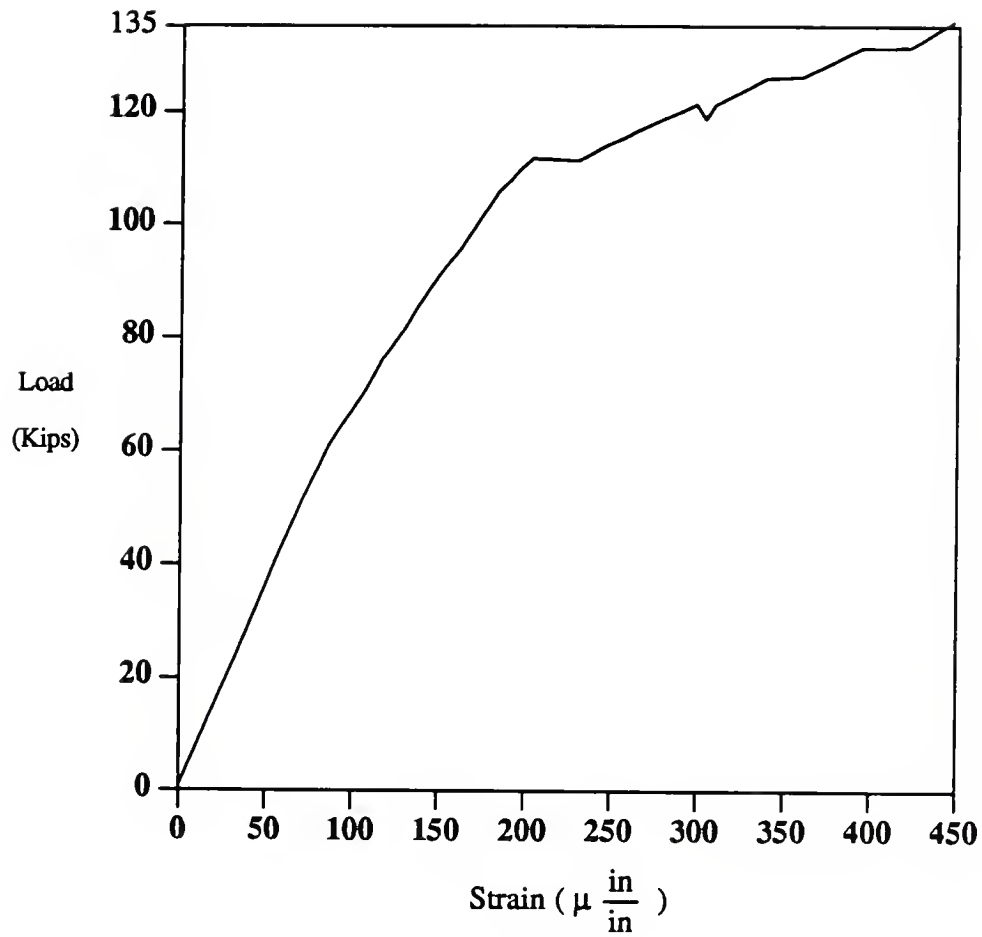


Figure 3.41 Surface Strain in Slab, at the Point Load
Beam with 50% Debonding (Specimen Set 1)

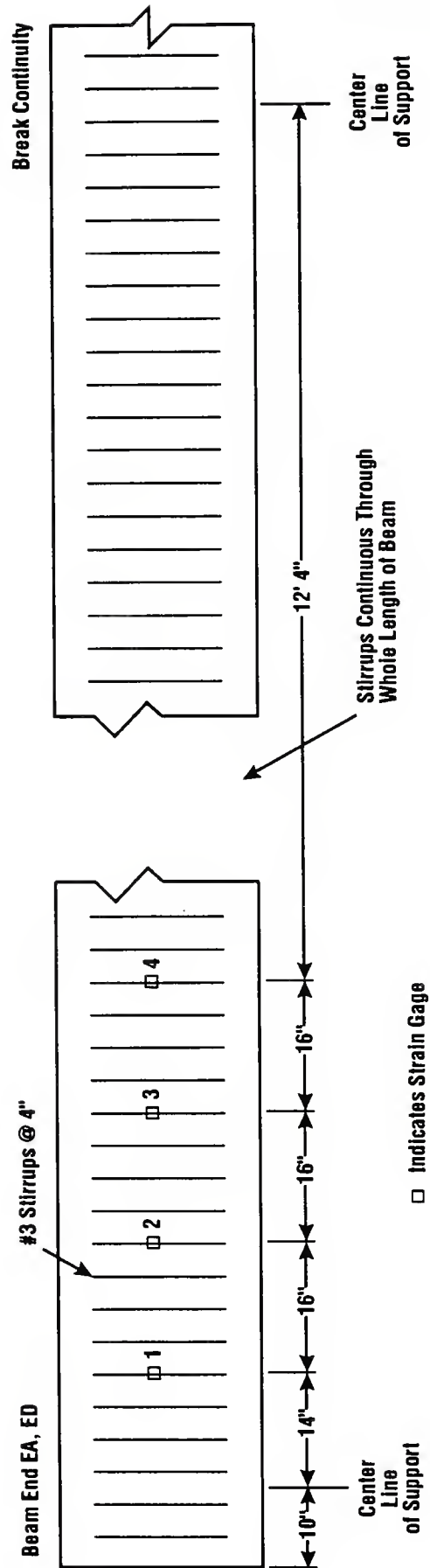


Figure (3.42) Location of Stirrup Reinforcement and Instrumentation for Specimen Sets 1 and 2.

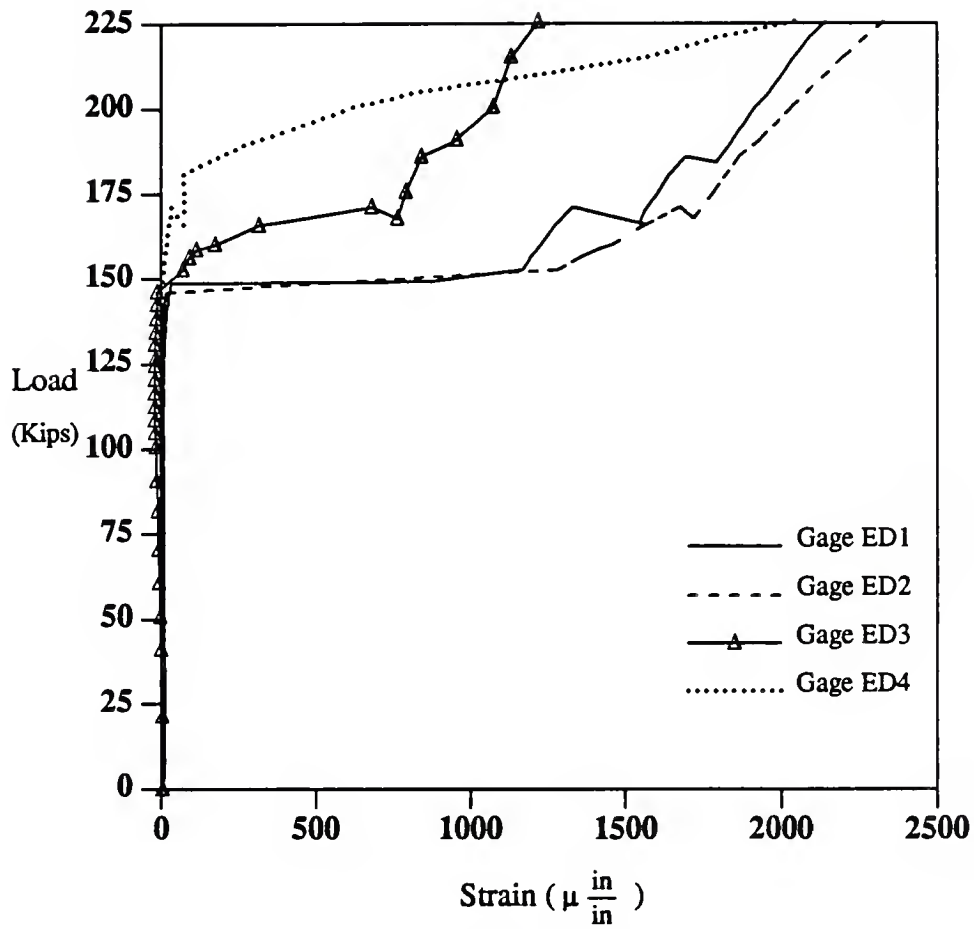


Figure 3.43 Stirrup Strain, Beam with 0% Debonding
(Specimen Set 1)

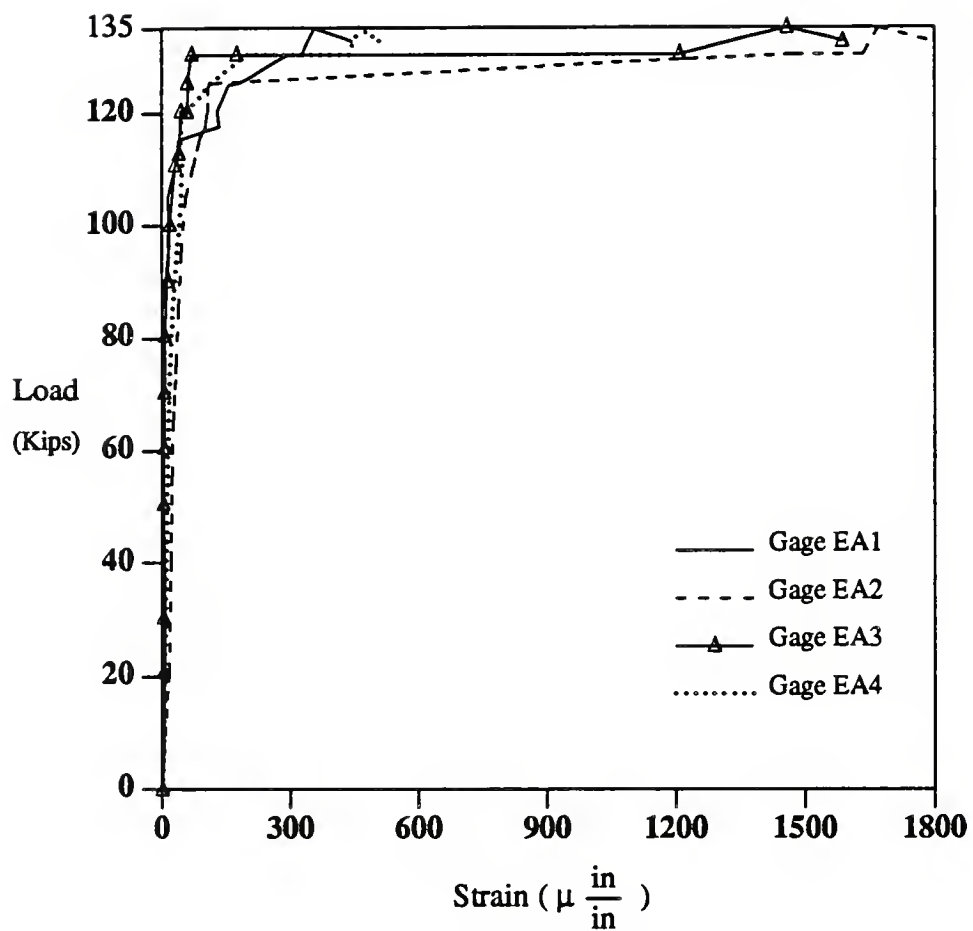
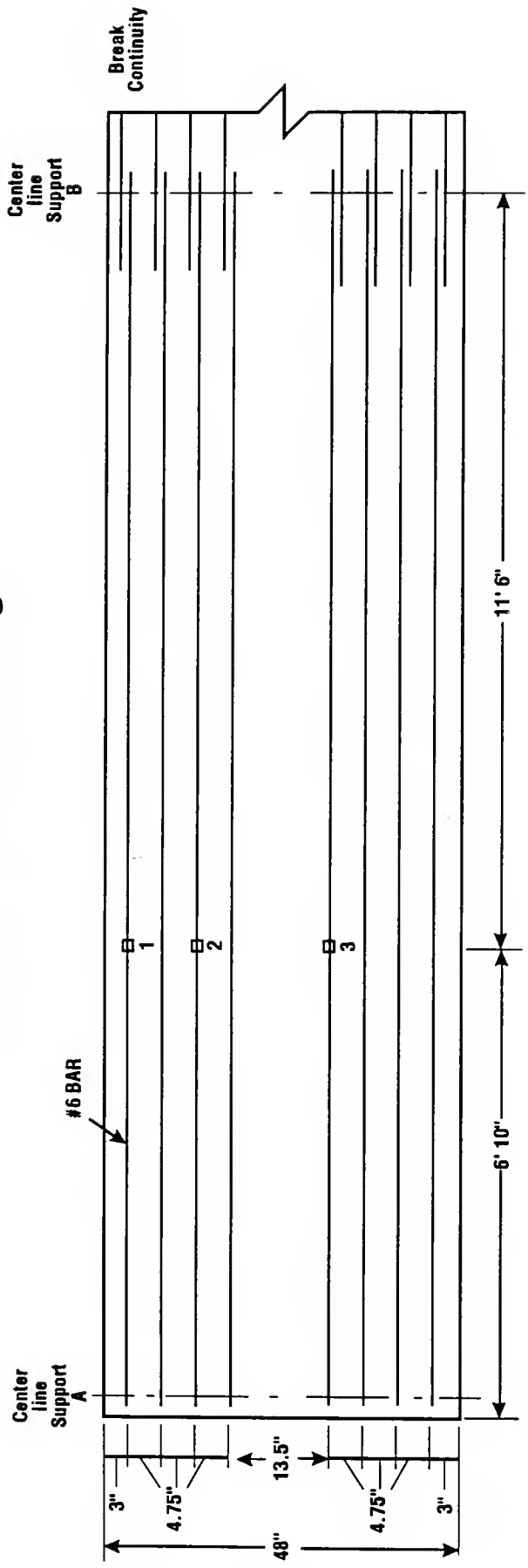
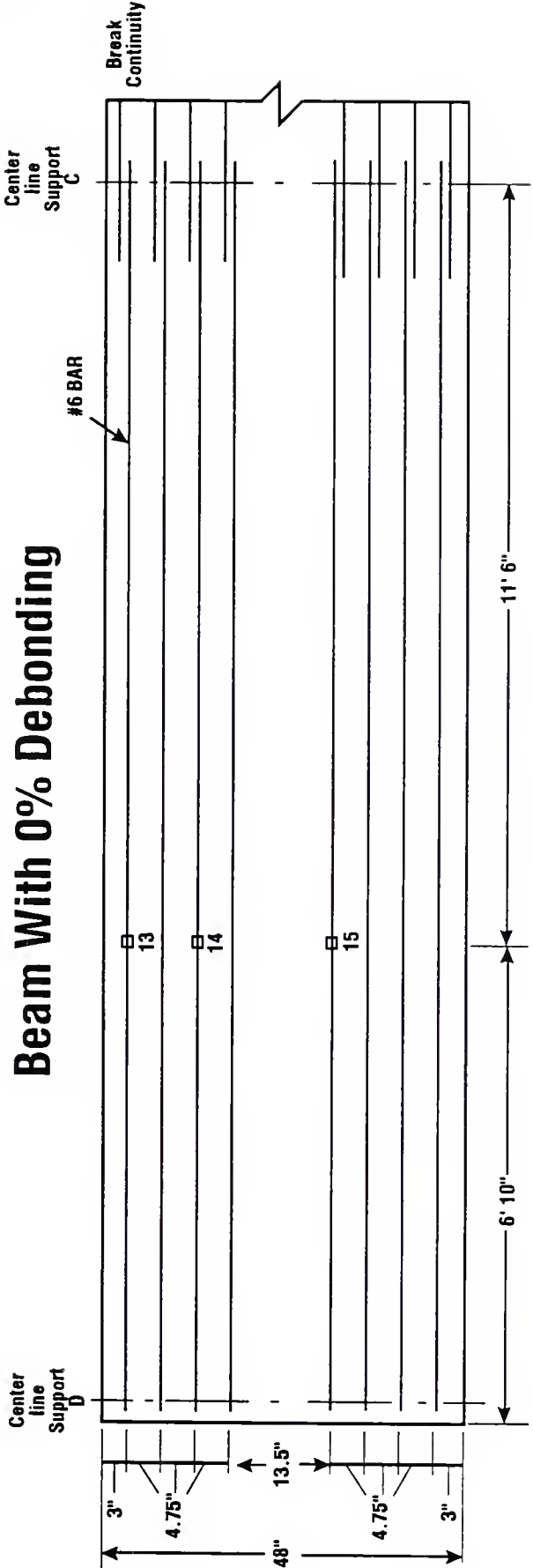


Figure 3.44 Stirrup Strain, Beam with 50% Debonding
(Specimen Set 1)

Beam With Debonding



Beam With 0% Debonding



□ Indicates Strain Gage Location

Figure (3.45) Slab Longitudinal Steel Instrumentation for Specimen Sets 1, 2 and 3.

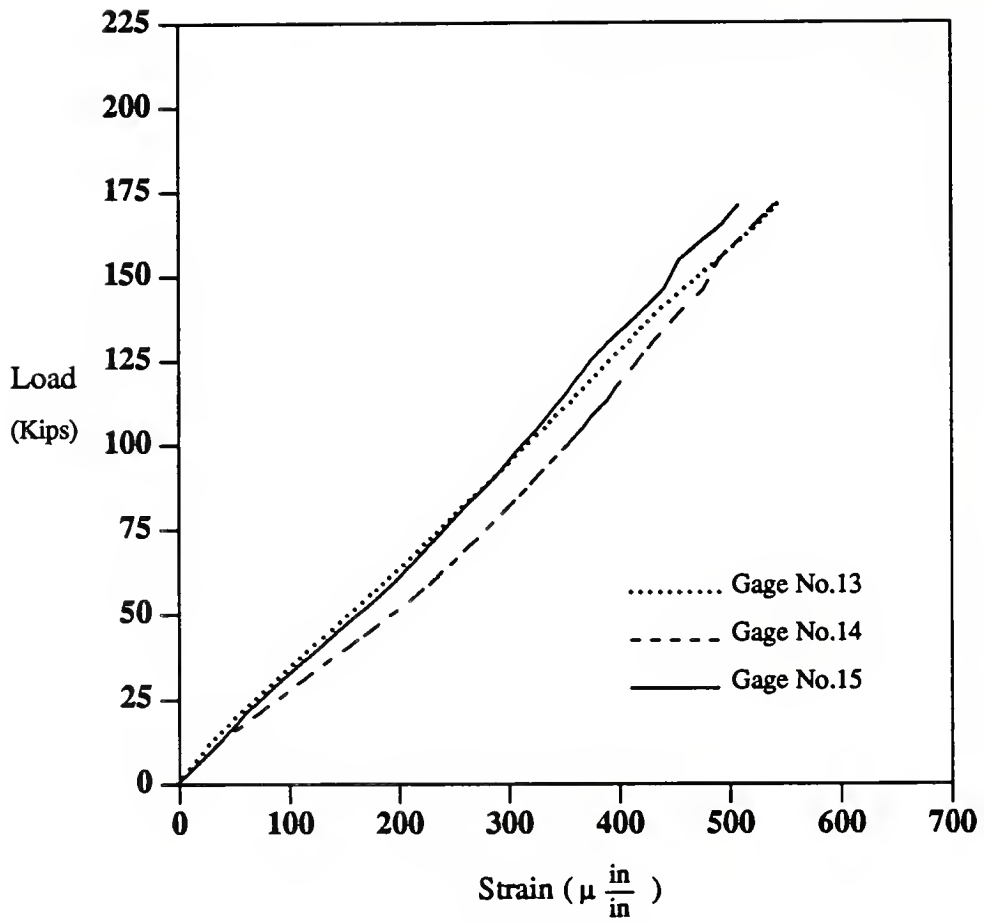


Figure 3.46 Strain in Slab Longitudinal Steel at 6.0 ft. From Support Beam with 0% Debonding (Specimen Set 1)

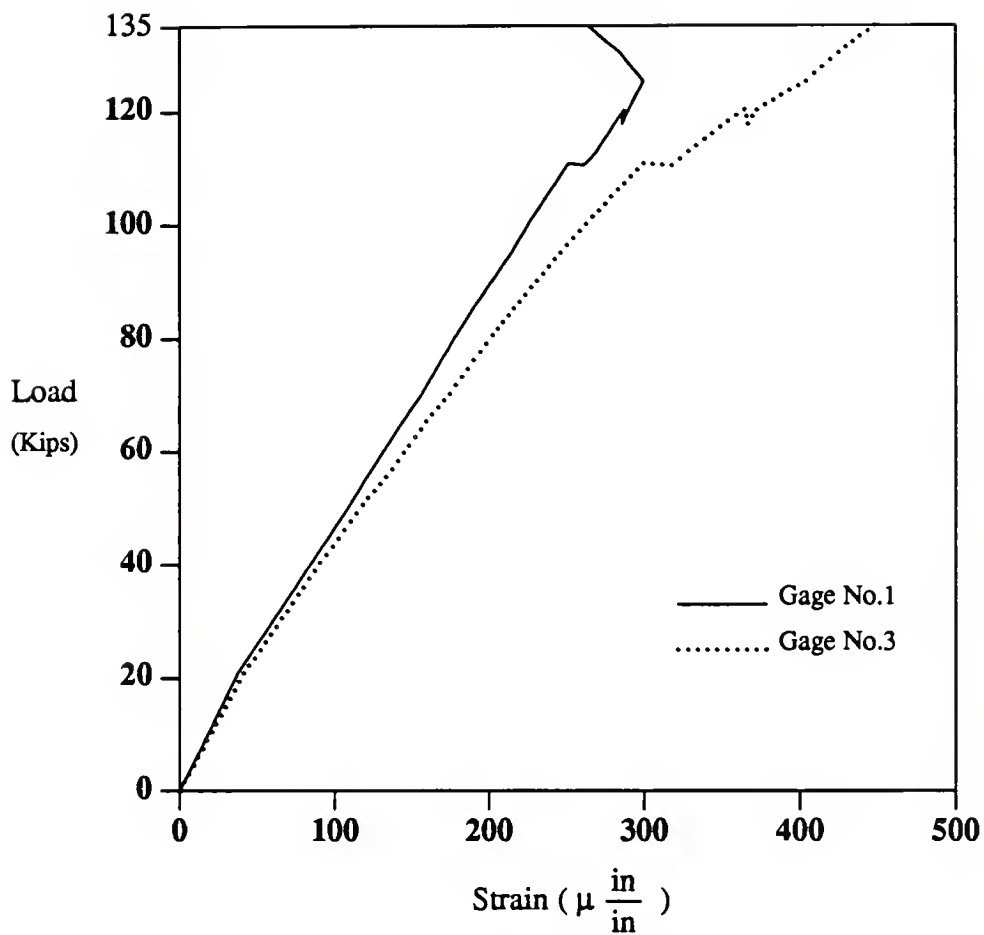


Figure 3.47 Strain in Slab Longitudinal Steel at 6 ft. from Support Beam with 50% Debonding (Specimen Set 1)

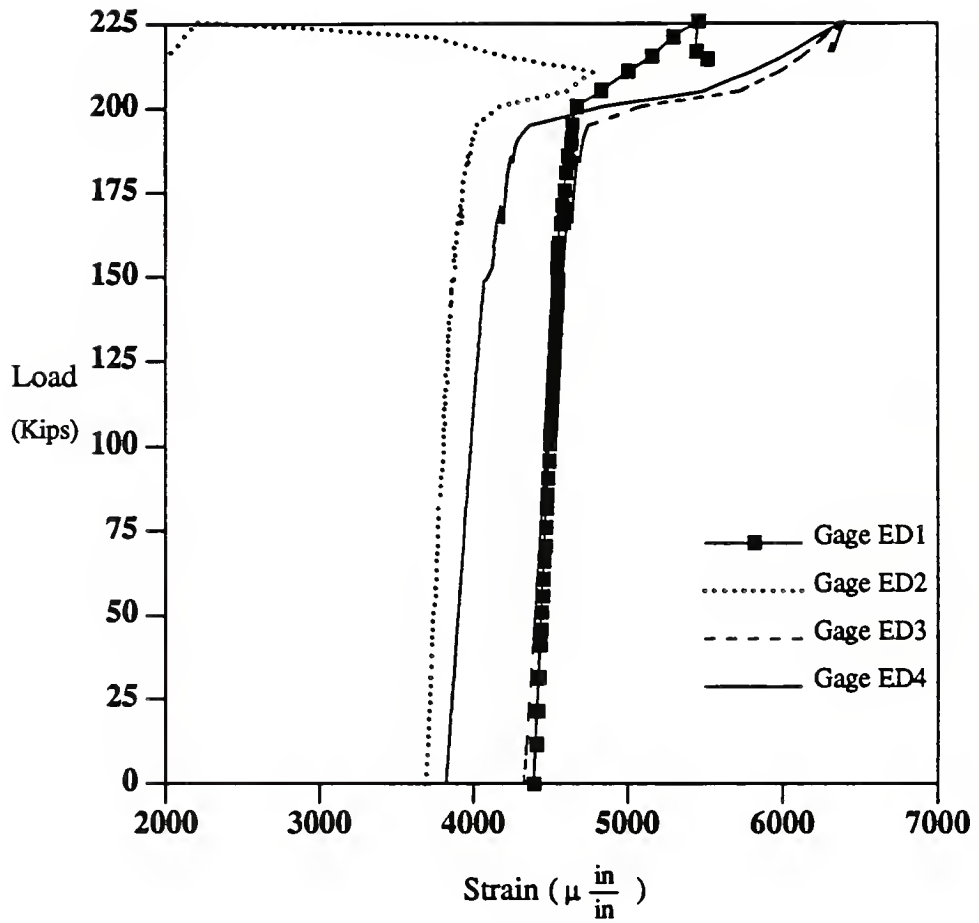


Figure 3.48 Strand Strains at 45 in. from Support
Beam with 0% Debonding (Specimen Set 1)

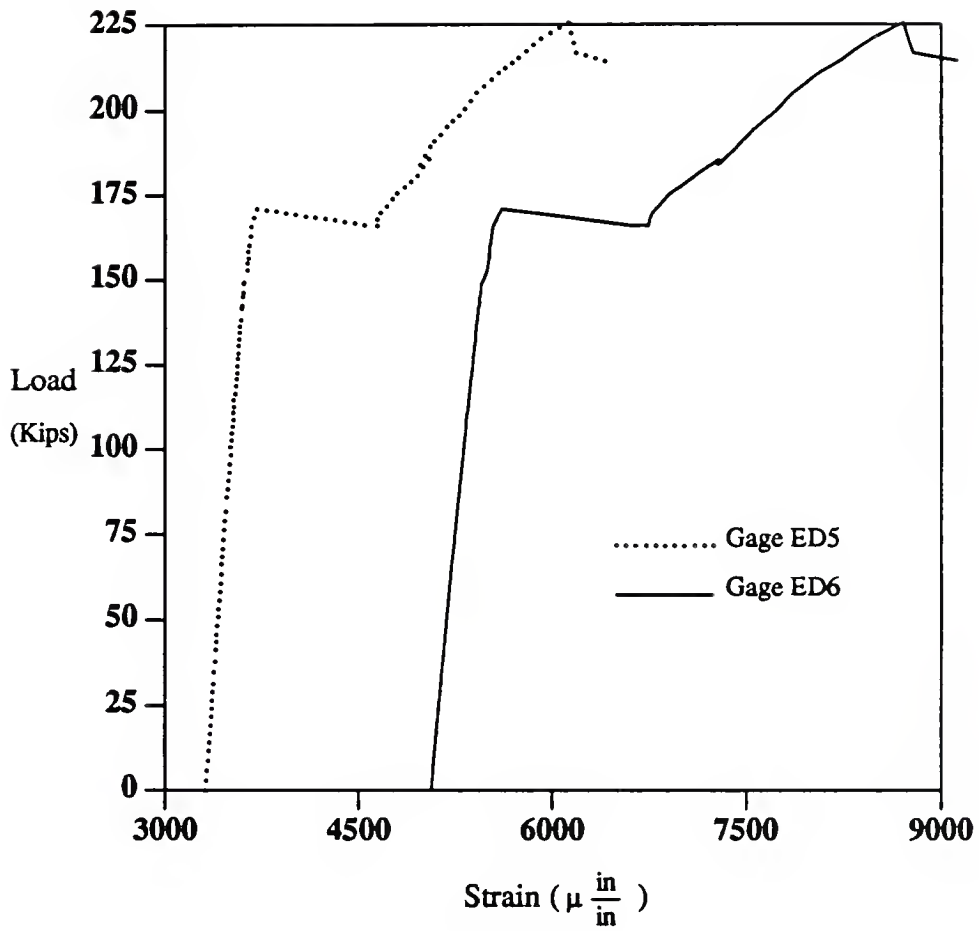


Figure 3.49 Strand Strains at 68 in. from Support
Beam with 0% Debonding (Specimen Set 1)

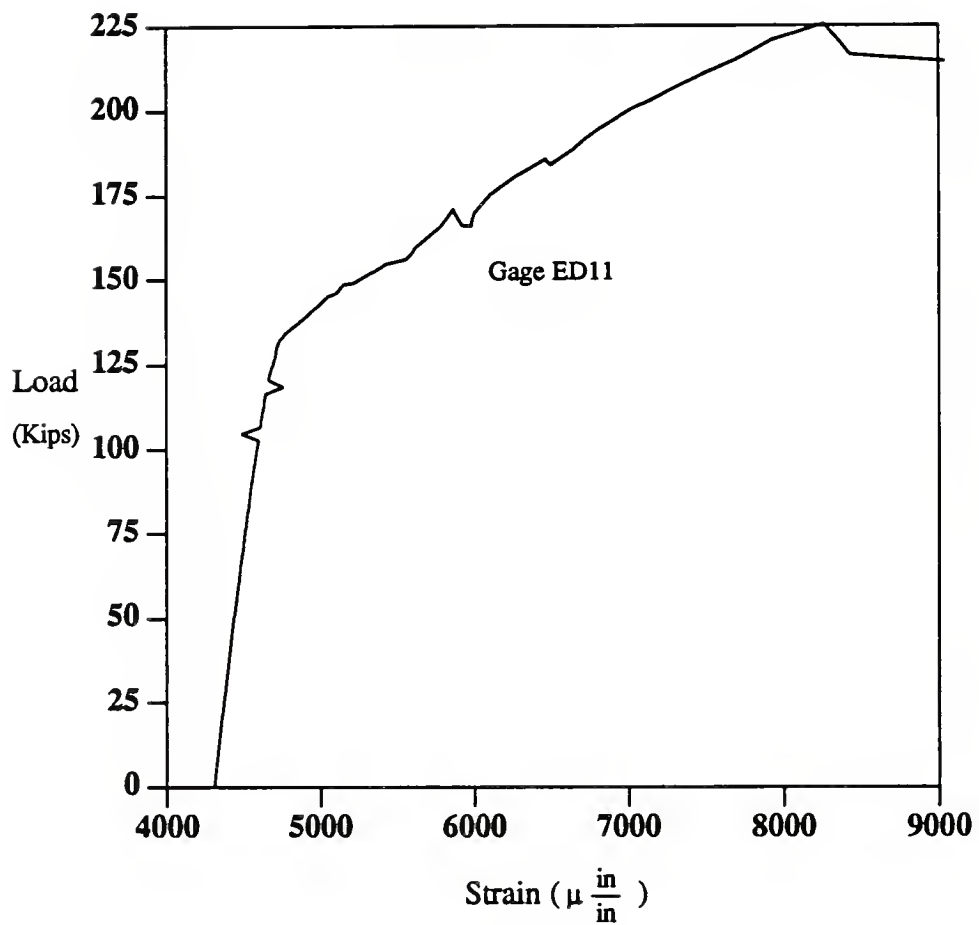


Figure 3.50 Strand Strains at 88 in. from Support
Beam with 0% Debonding (Specimen Set 1)

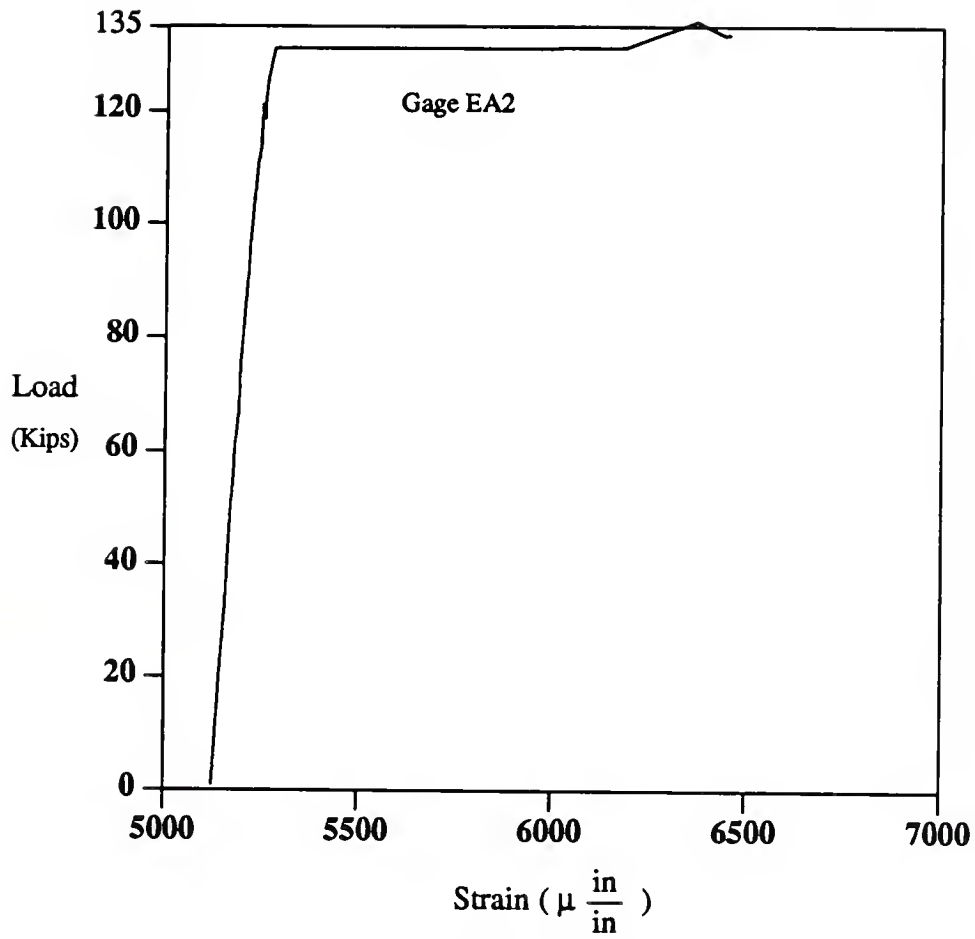


Figure 3.51 Strand Strains at First Debonding Point
Beam with 50% Debonding (Specimen Set 1)

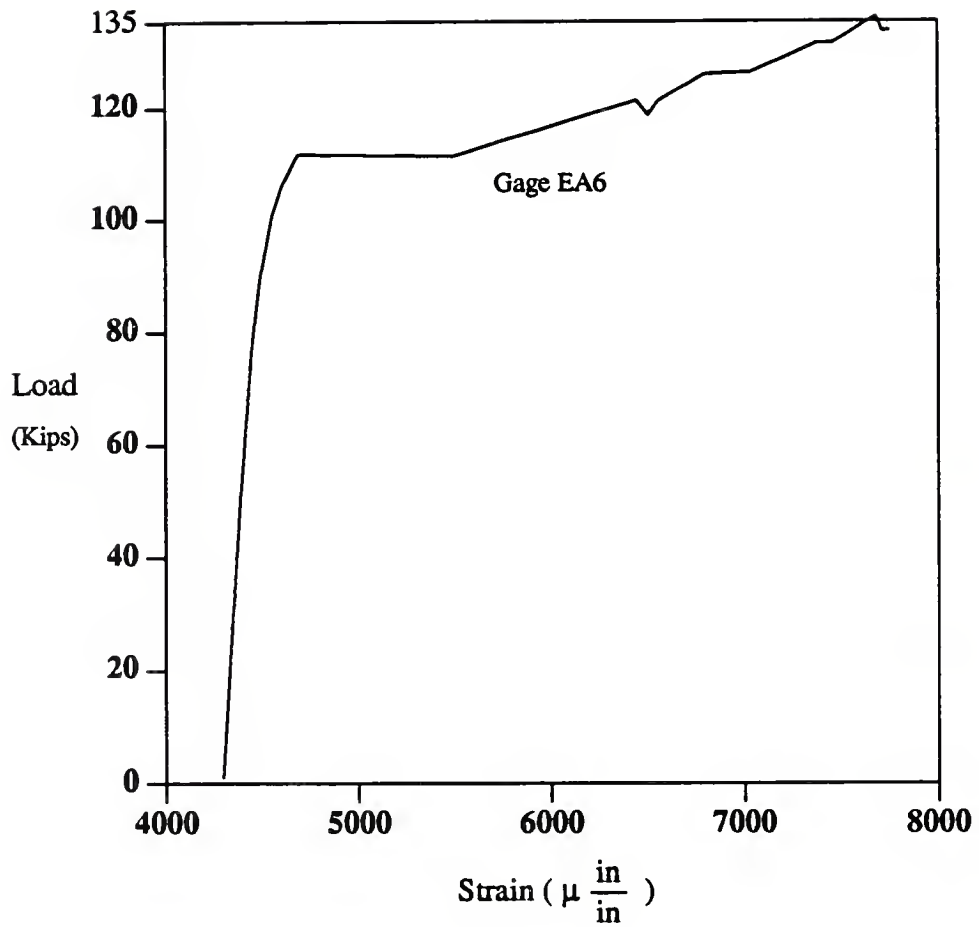


Figure 3.52 Strand Strains at Second Debonding Point
Beam with 50% Debonding (Specimen Set 1)

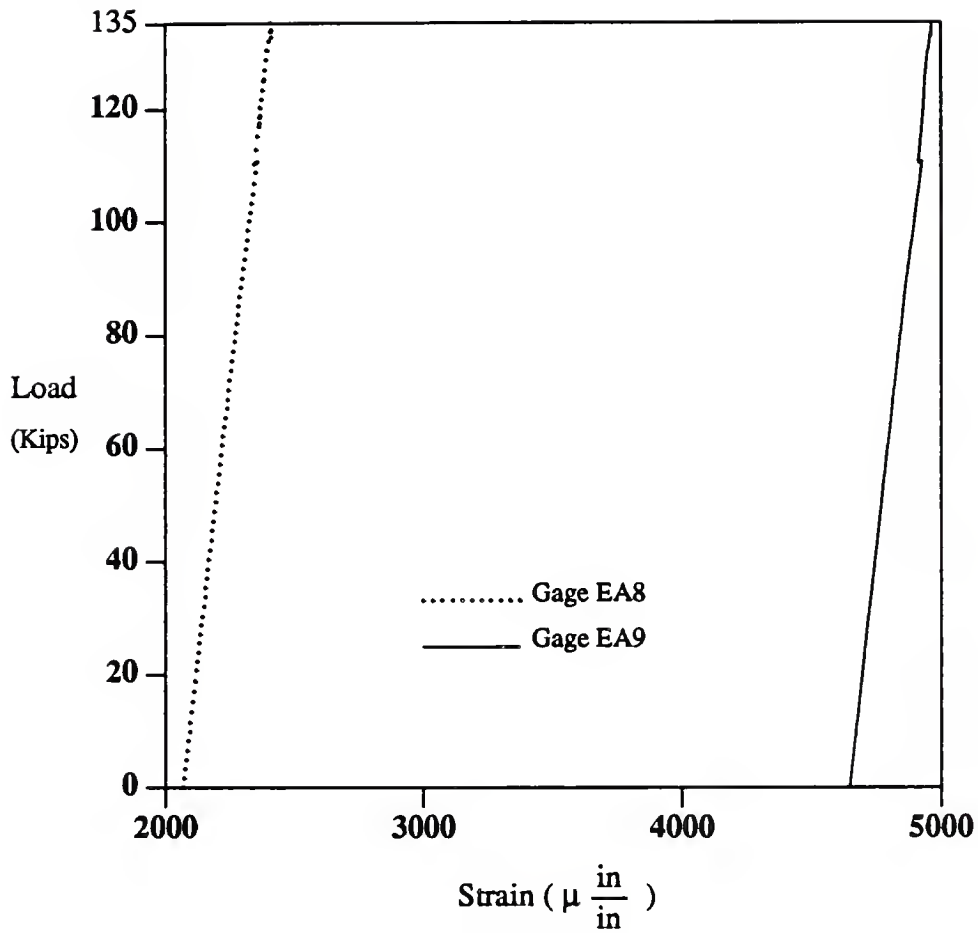


Figure 3.53 Strand Strains at 82 in. from Support
Beam with 50% Debonding (Specimen Set 1)

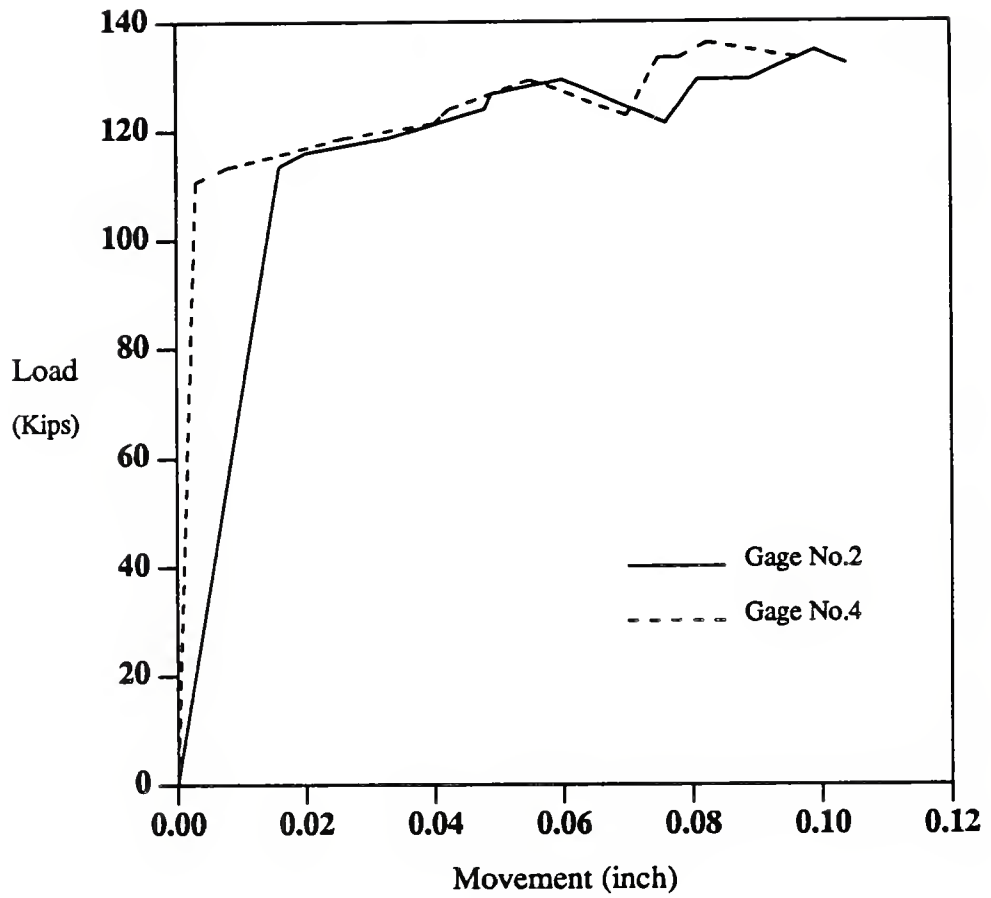
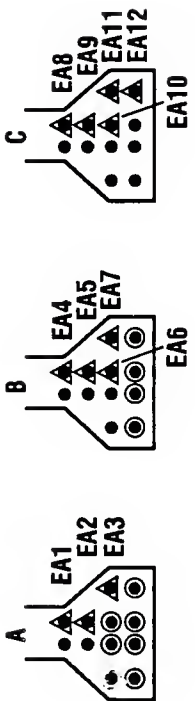
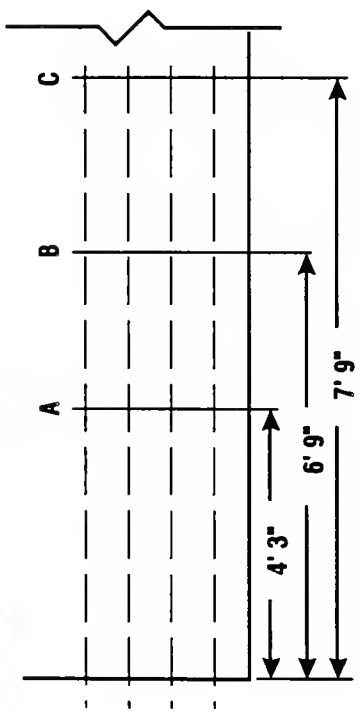


Figure 3.54 Strand Movement, Beam with 50% Debonding
Specimen Set 1

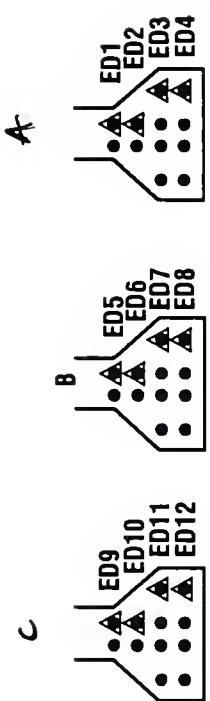
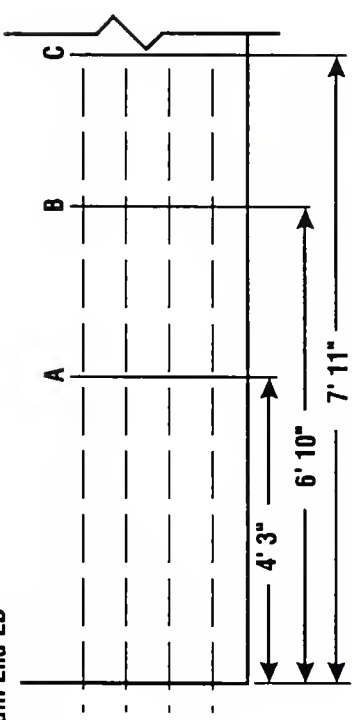
Beam With Debonding

Beam End EA 50%



Beam With 0% Debonding

Beam End ED



△ Indicates Instrumented Strand ○ Indicates Bond Breaks

Figure (3.55) Strand Debonding Scheme and Instrumentation for Specimen Set 2.

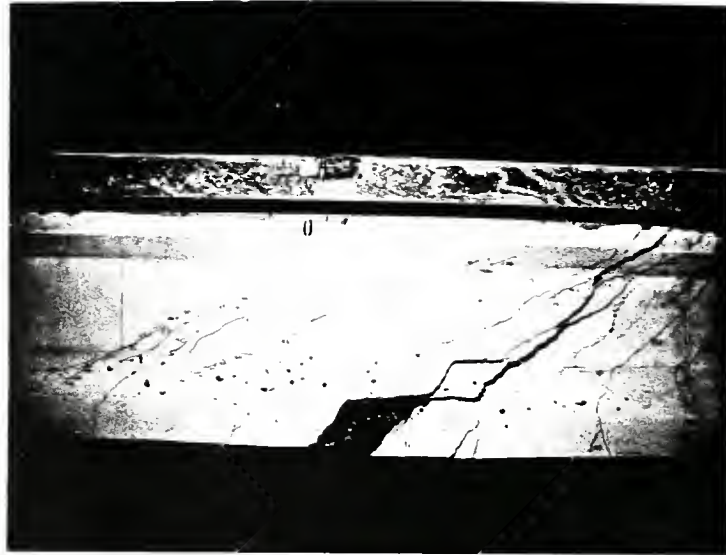


Figure 3.56 Crack Pattern of 0% Debonded Beam at Failure
Specimen Set 2



Figure 3.57 Crack Pattern of 50% Debonded Beam at Failure
Specimen Set 2

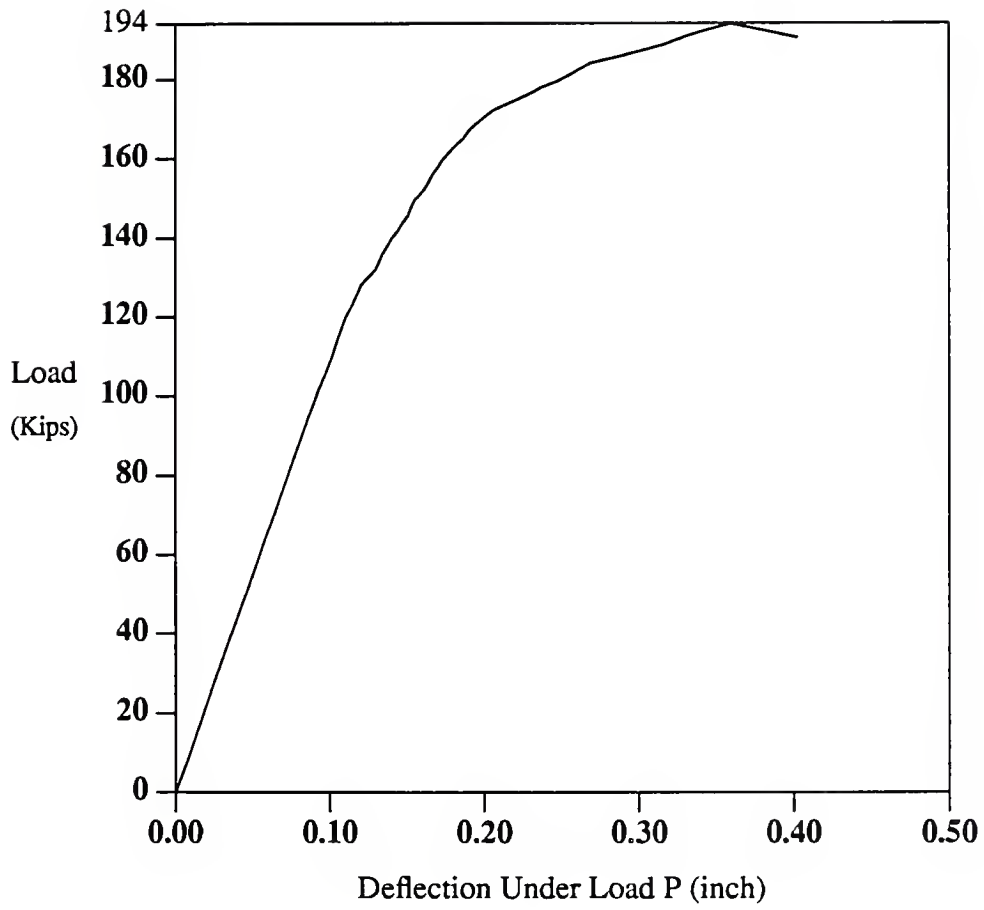


Figure 3.58 Load-Deflection Relationship, Beam with 0% Debonding
(Specimen Set 2)

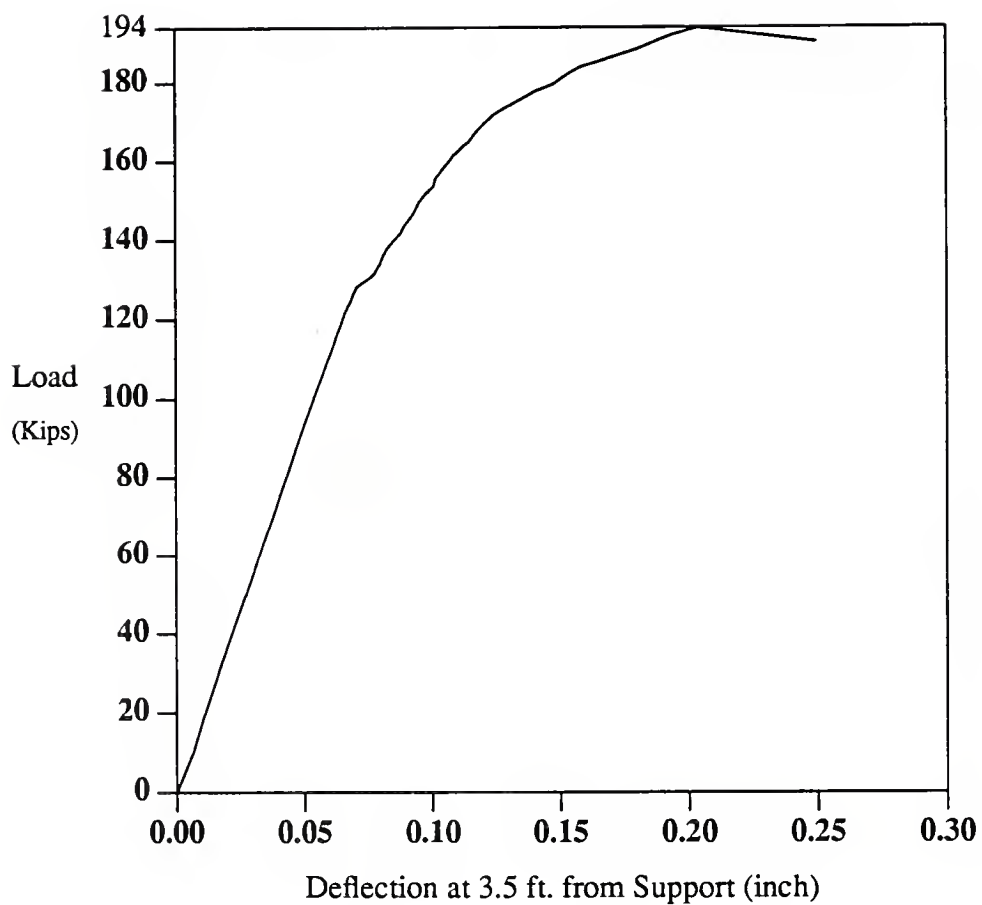


Figure 3.59 Load-Deflection Relationship, Beam with 0% Debonding
(Specimen Set 2)

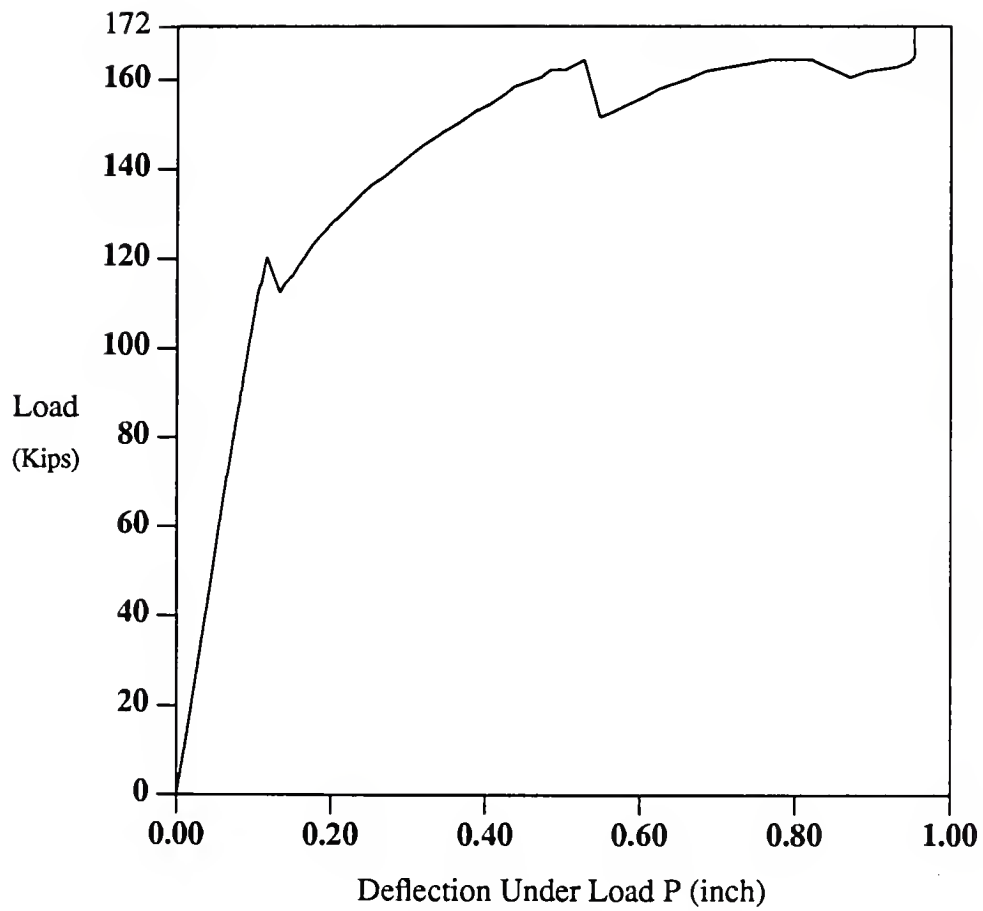


Figure 3.60 Load-Deflection Relationship, Beam with 50% Debonding
(Specimen Set 2)

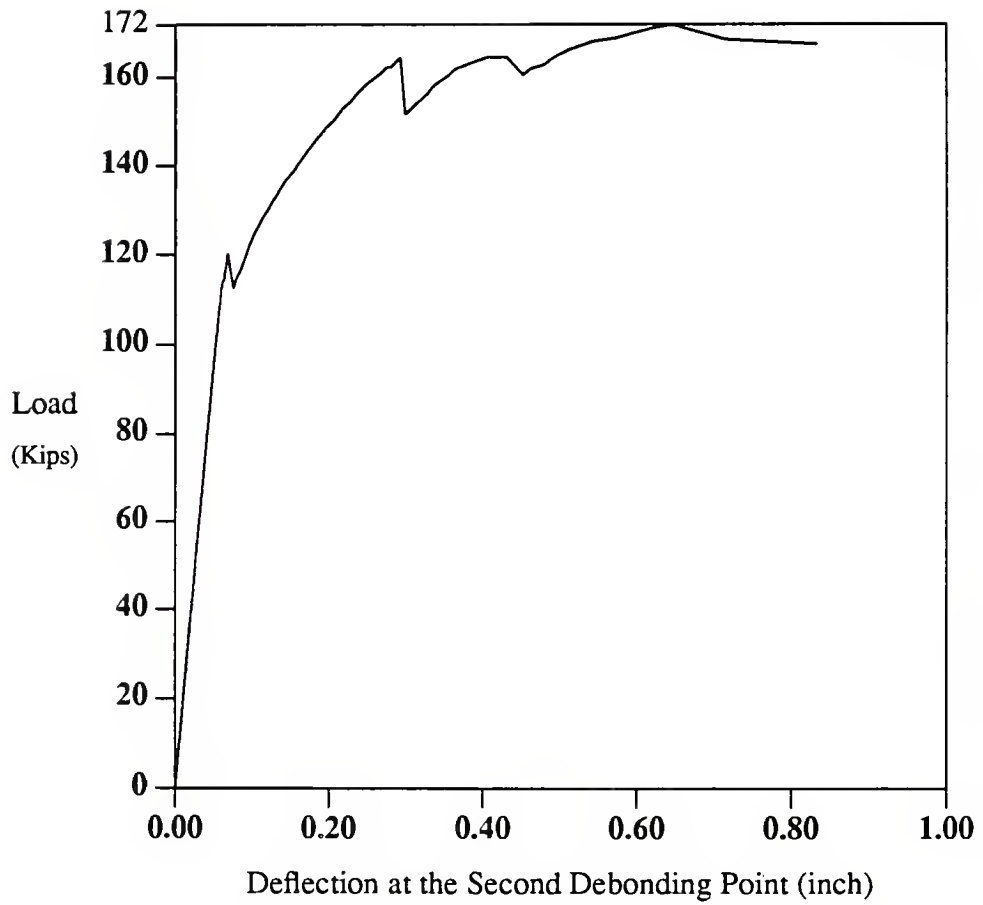


Figure 3.61 Load-Deflection Relationship, Beam with 50% Debonding (Specimen Set 2)

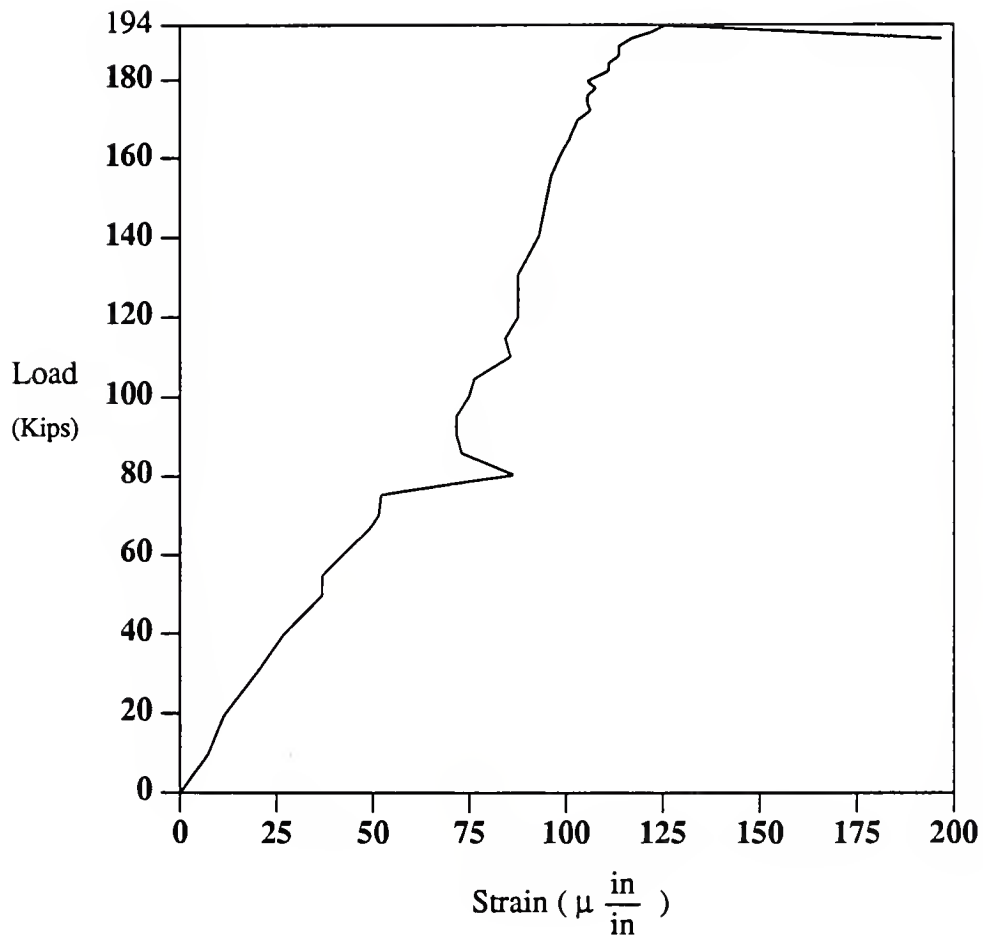


Figure 3.62 Surface Strain in Slab, at 3.5 ft. from Support Beam with 0% Debonding (Specimen Set 2)

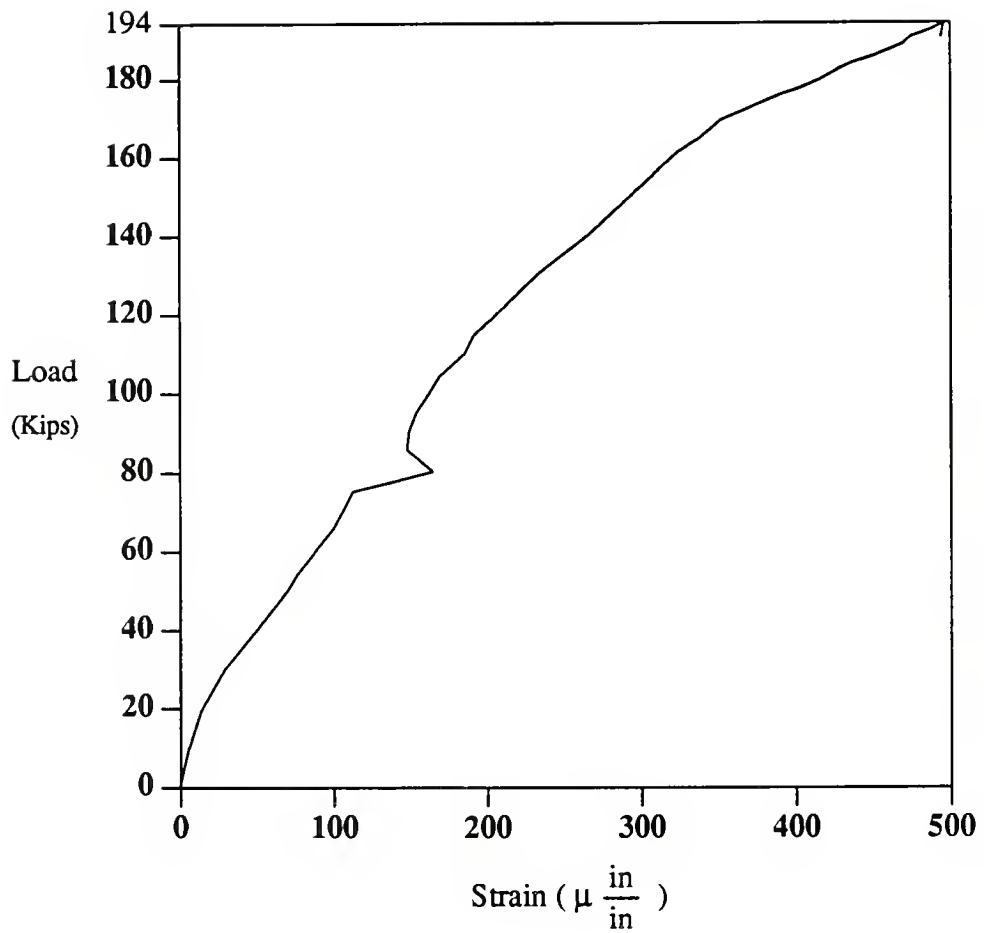


Figure 3.63 Surface Strain in Slab at the Point Load Beam with 0% Debonding (Specimen Set 2)

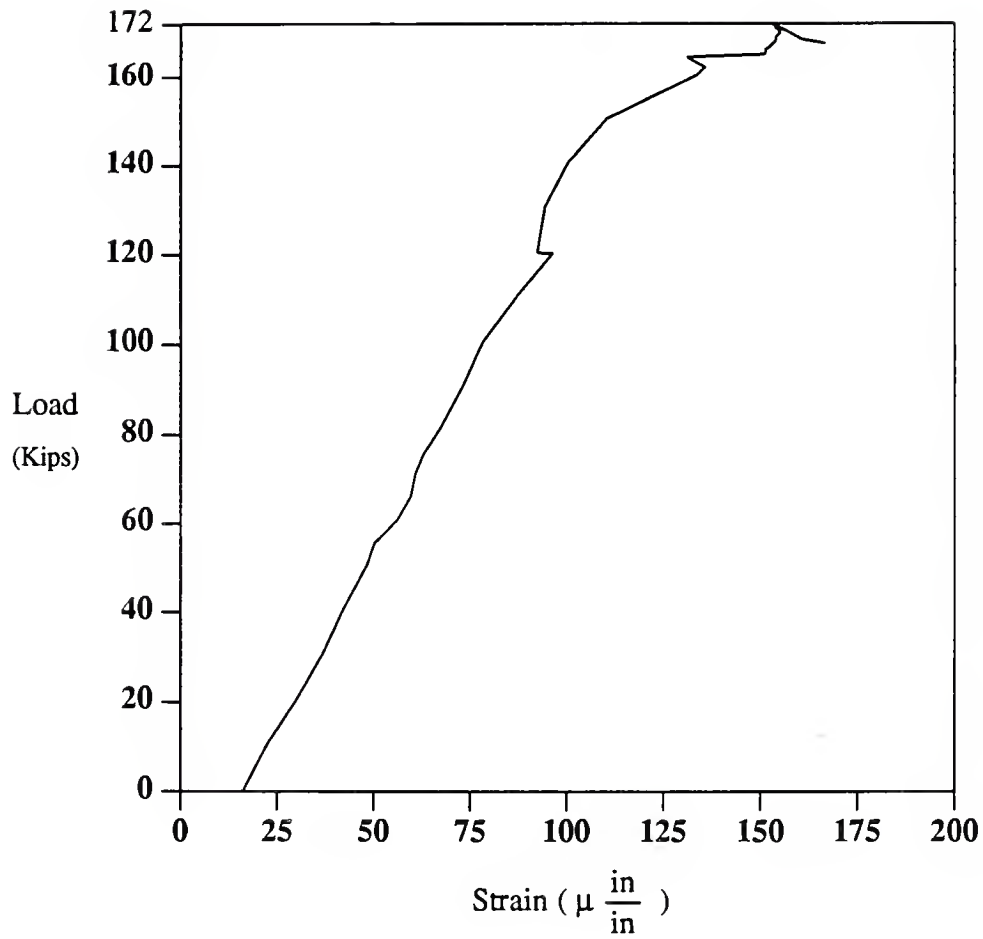


Figure 3.64 Surface Strain in Slab at First Debonding Point
Beam with 50% Debonding (Specimen Set 2)

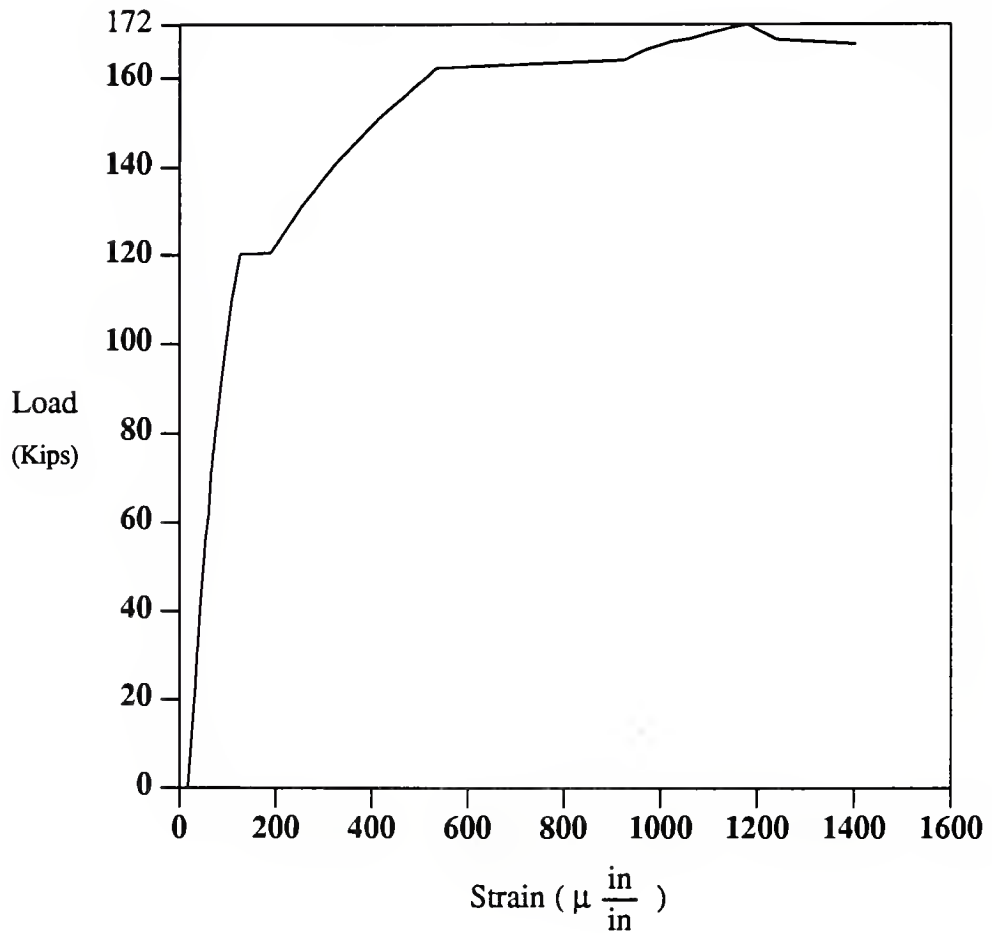


Figure 3.65 Surface Strain in Slab. at the Point Load
Beam with 50% Debonding (Specimen Set 2)

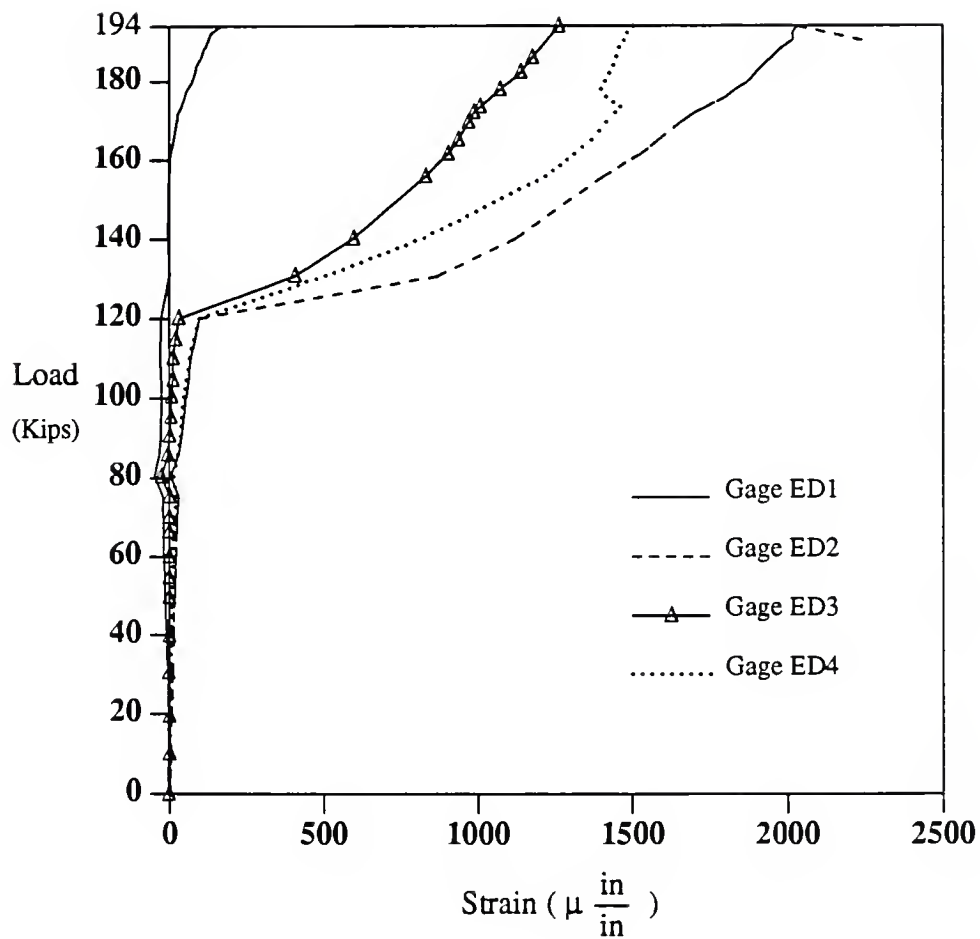


Figure 3.66 Stirrup Strain, Beam with 0% Debonding
(Specimen Set 2)

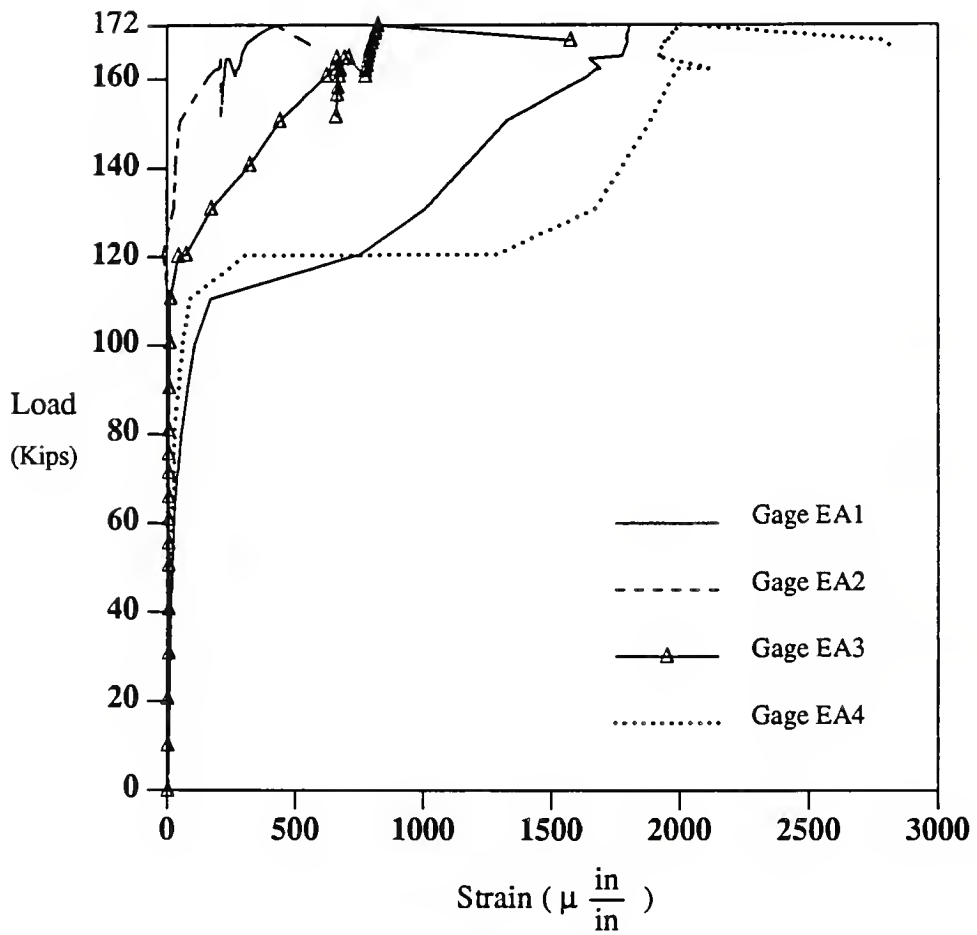


Figure 3.67 Stirrup Strain, Beam with 50% Debonding
(Specimen Set 2)

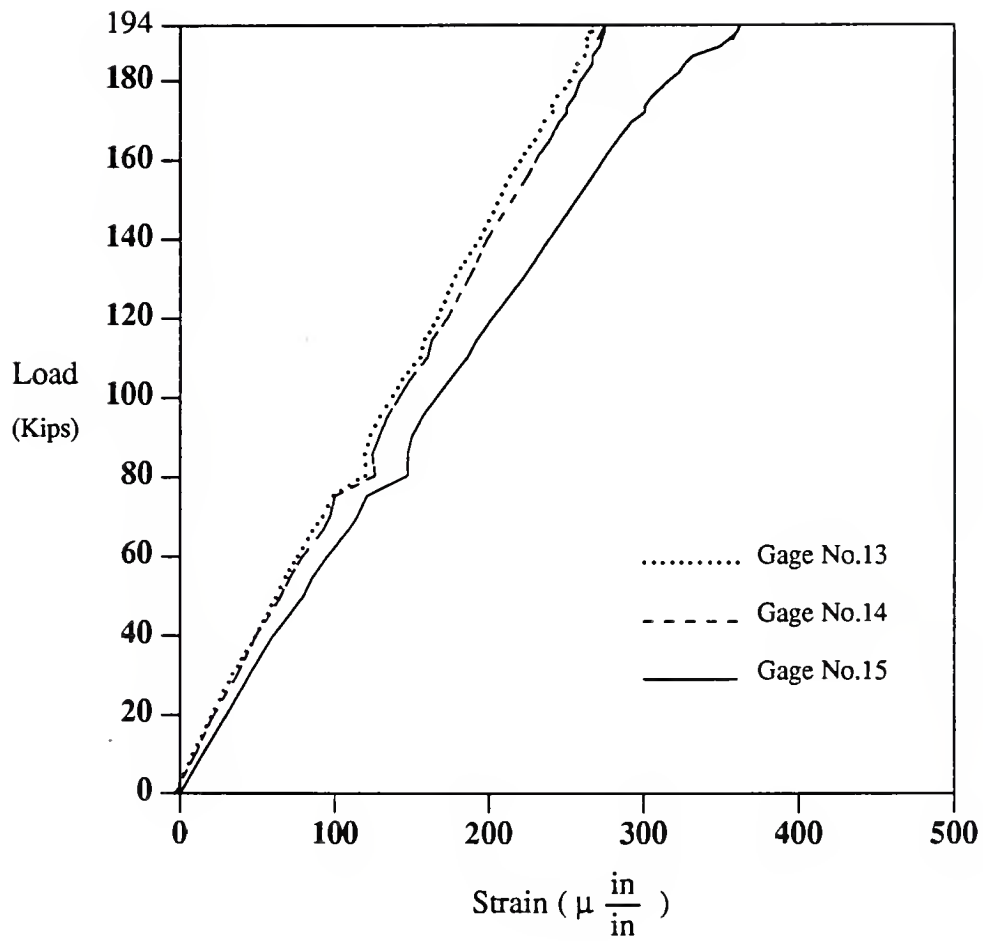


Figure 3.68 Strain in Slab Longitudinal Steel, at 6.0 ft. from Support
Beam with 0% Debonding (Specimen Set 2)

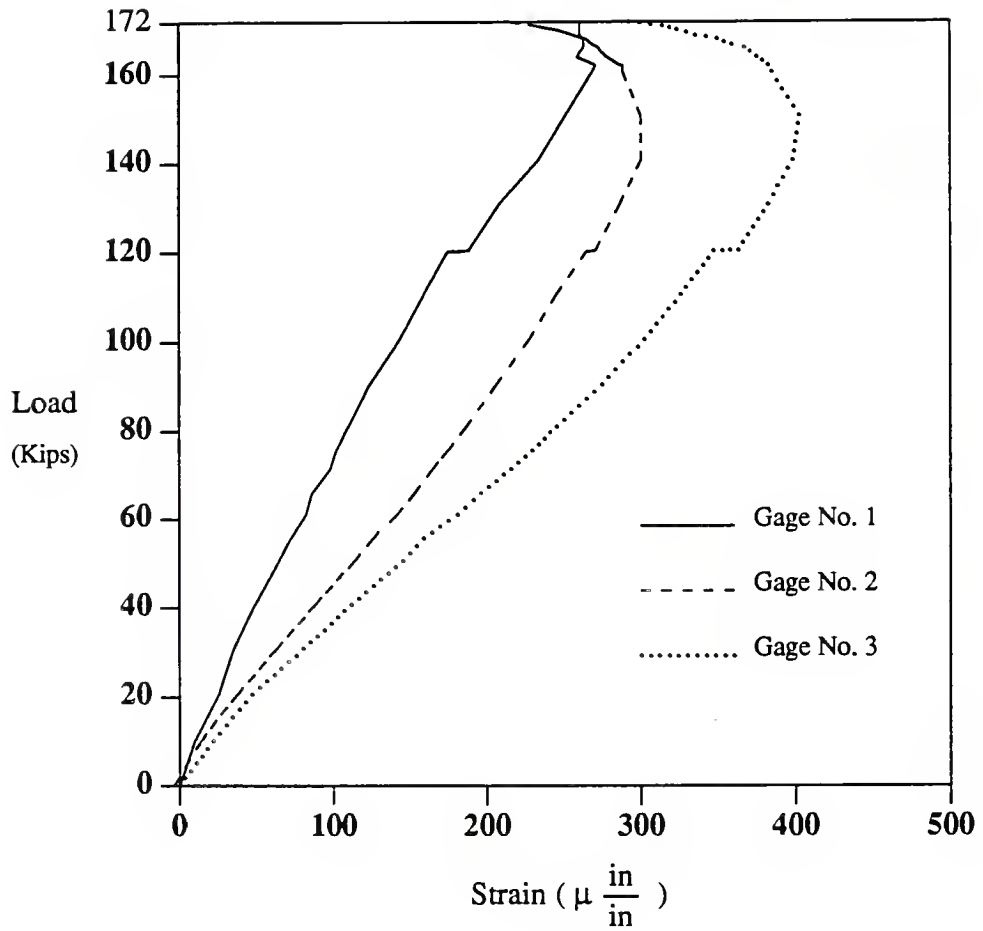


Figure 3.69 Strain in Slab Longitudinal Steel, 6.0 ft. from Support Beam with 50% Debonding (Specimen Set 2)

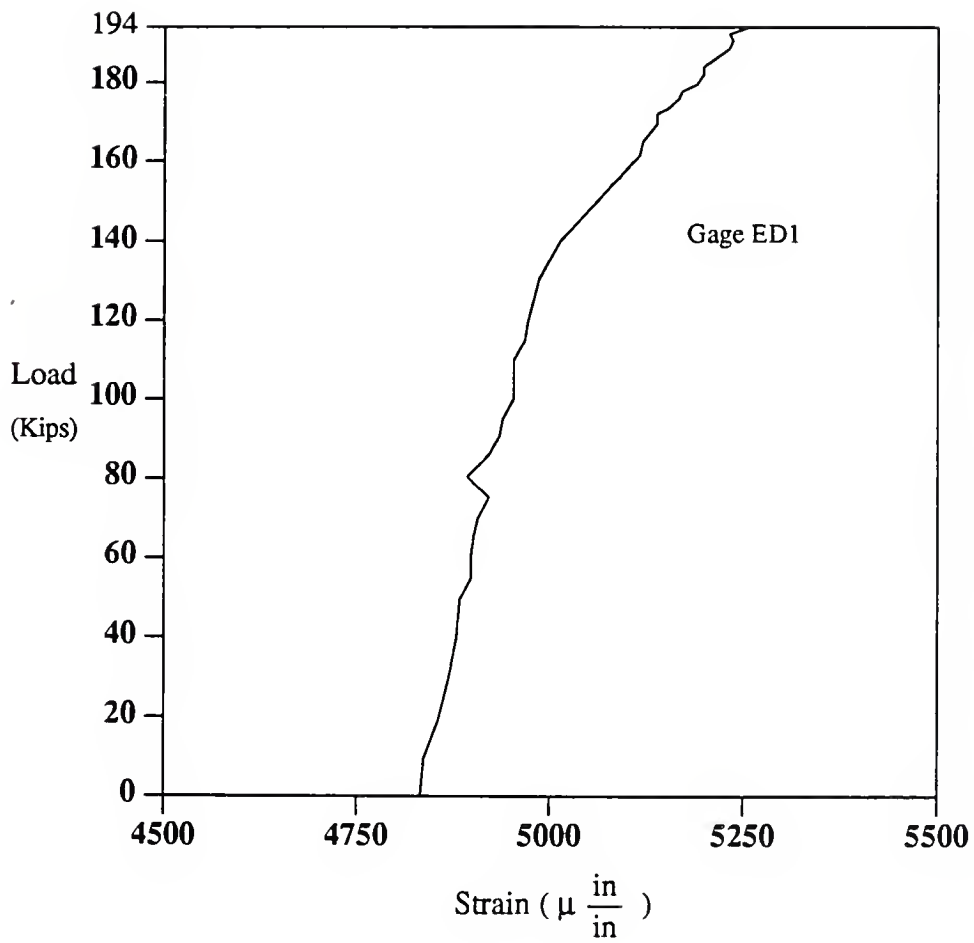


Figure 3.70 Strand Strain at 41 in. from Support
Beam with 0% Debonding (Specimen Set 2)

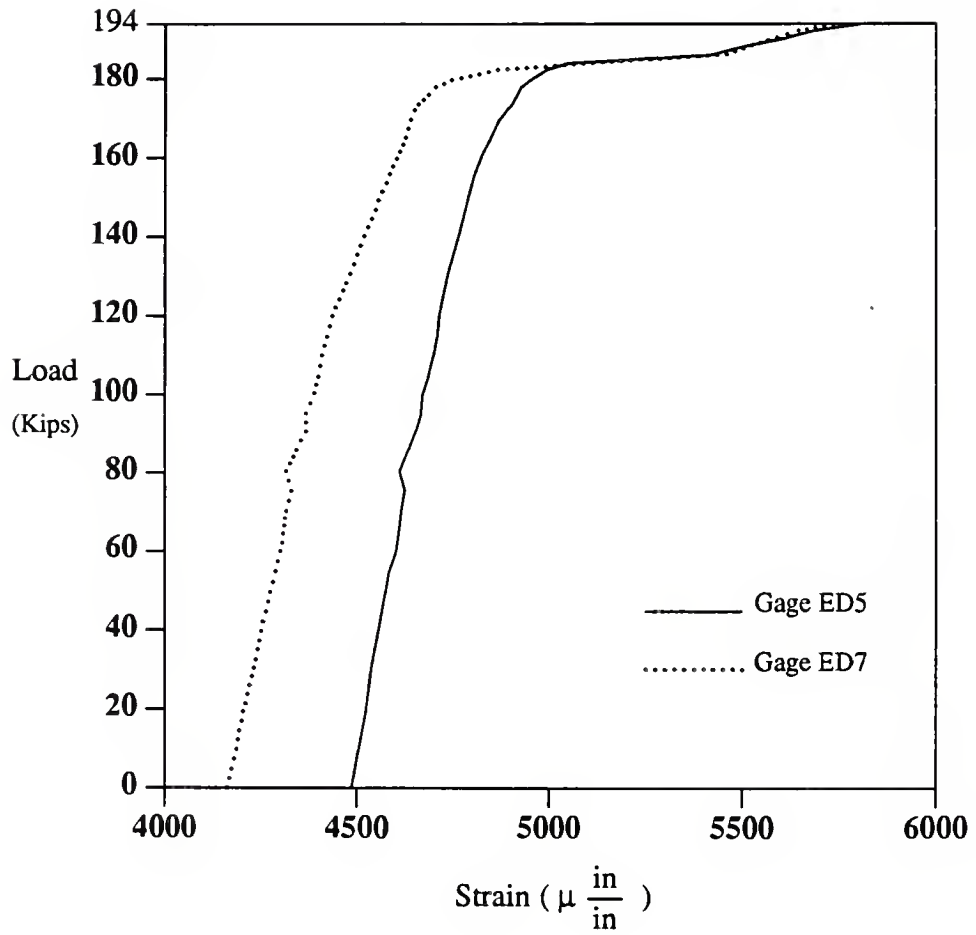


Figure 3.71 Strand Strain at 6 ft. from Support
Beam with 0% Debonding (Specimen Set 2)

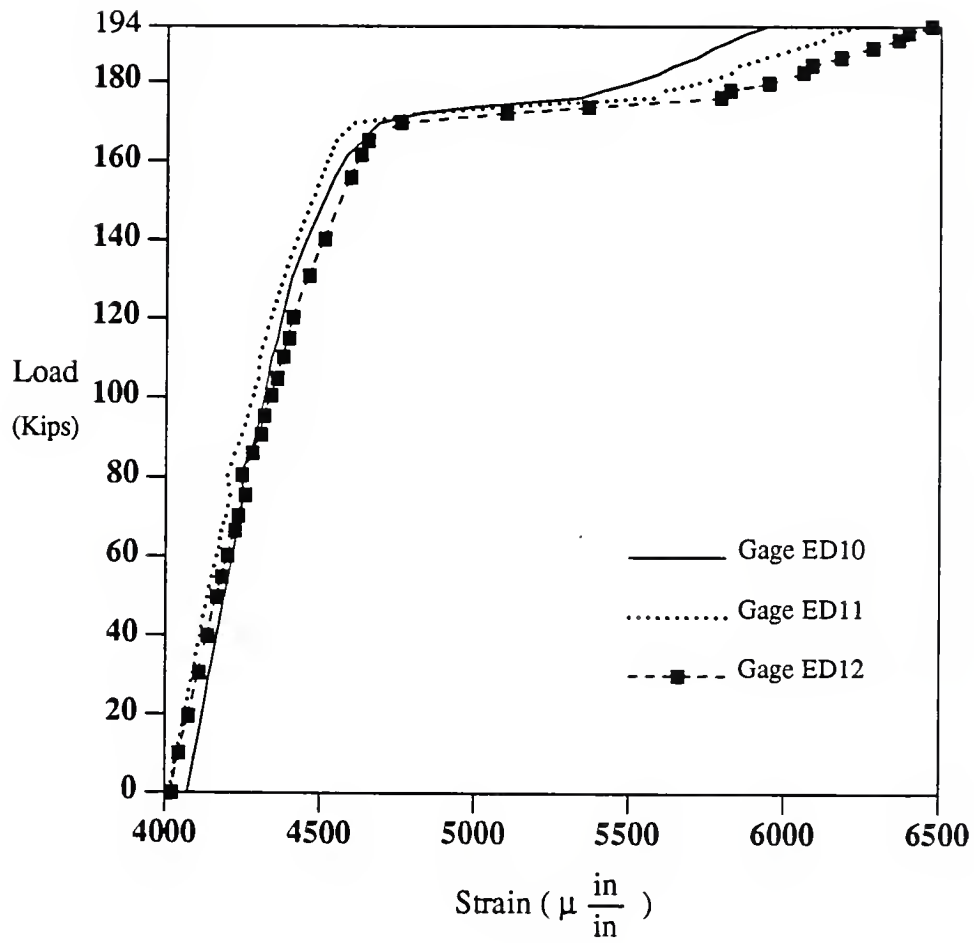


Figure 3.72 Strand Strain at 85 in. from Support
Beam with 0% Debonding (Specimen Set 2)

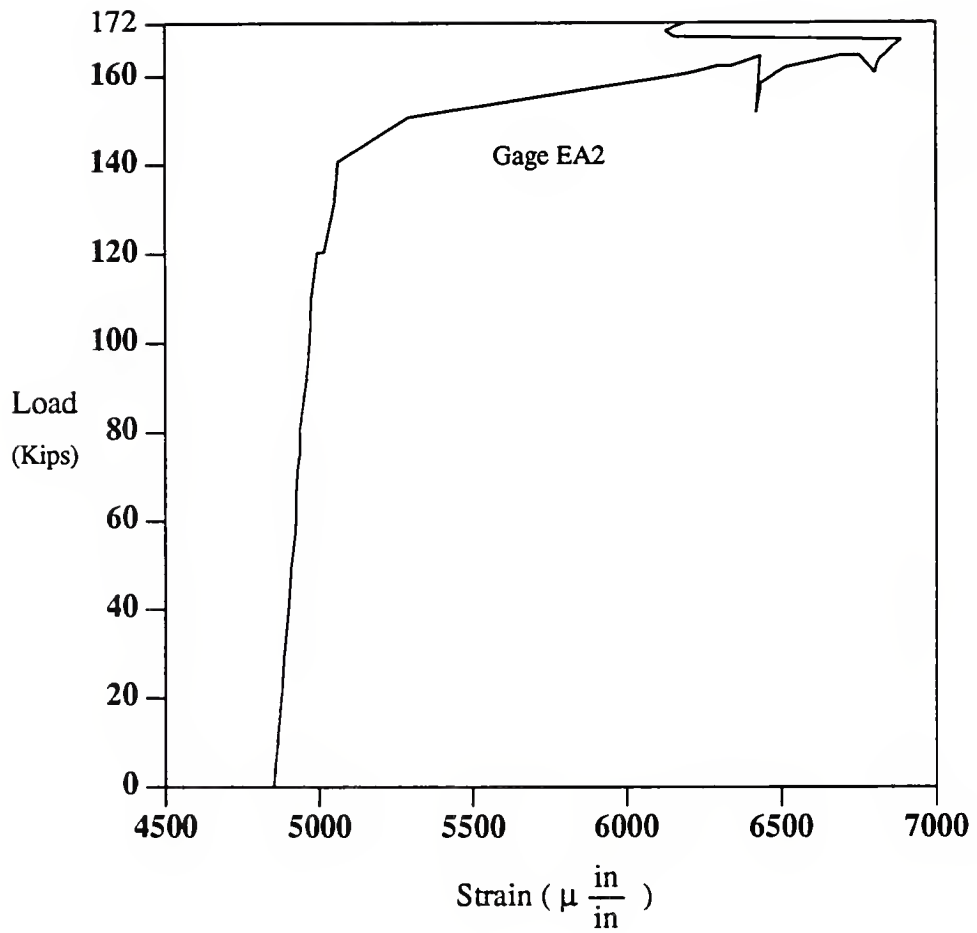


Figure 3.73 Strand Strain at First Debonding Point
Beam with 50% Debonding (Specimen Set 2)

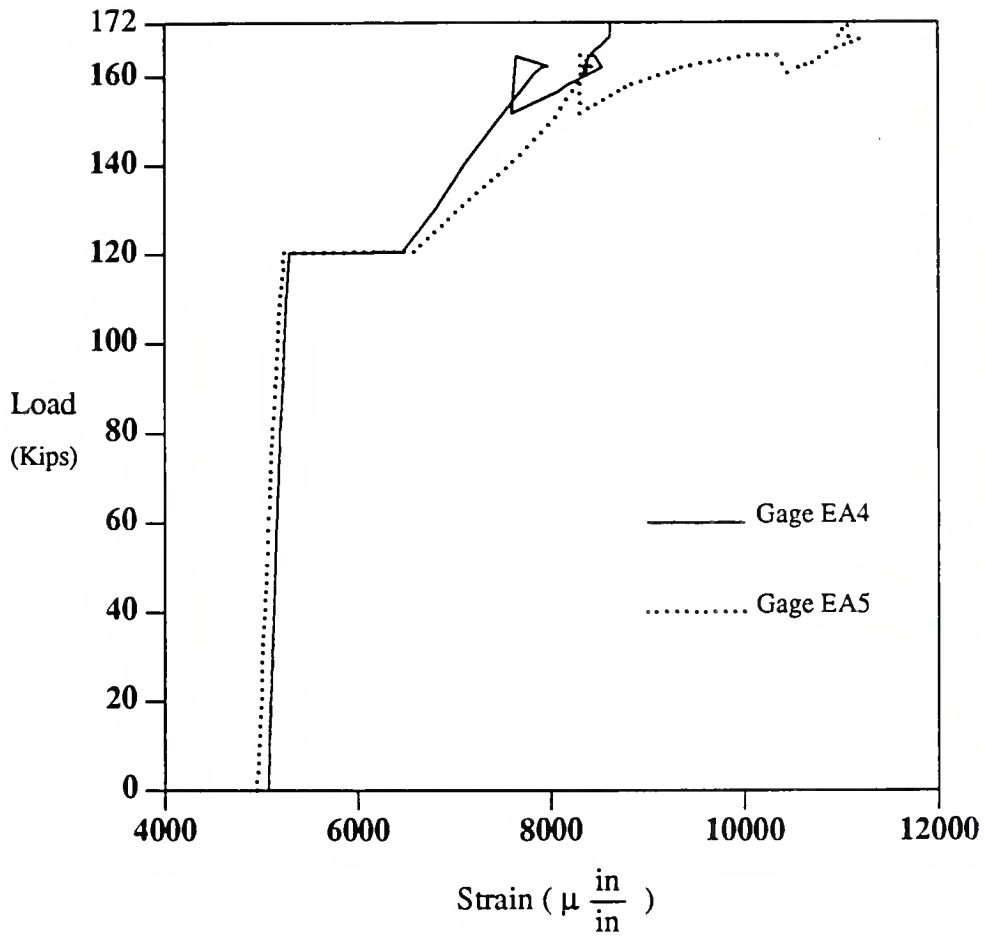


Figure 3.74 Strand Strains at Second Debonding Point
Beam with 50% Debonding (Specimen Set 2)

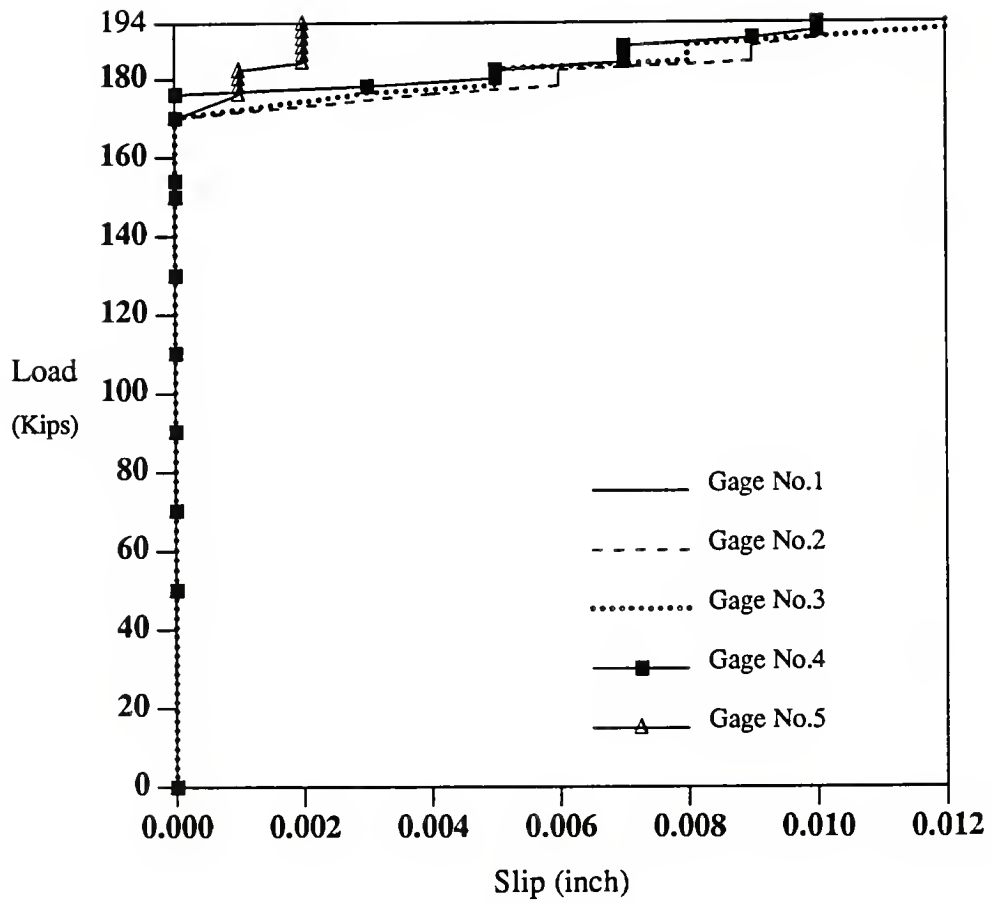


Figure 3.75 Strand Slip, Beam with 0% Debonding
Specimen Set 2

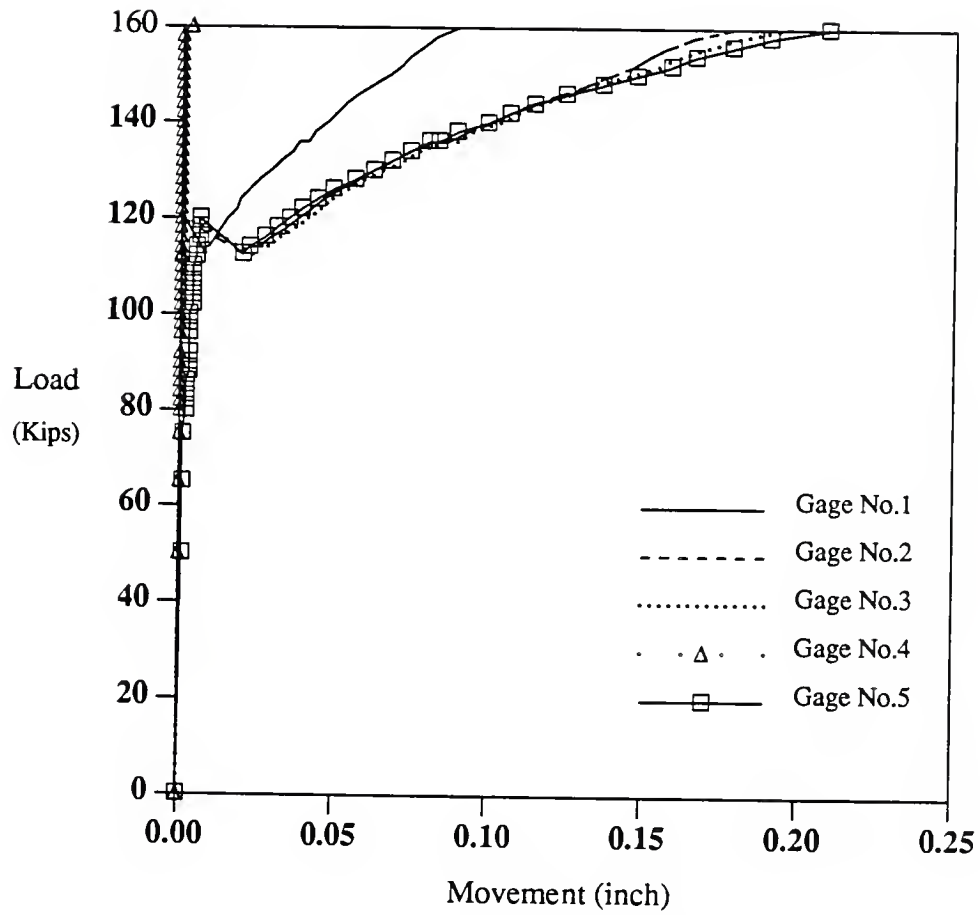
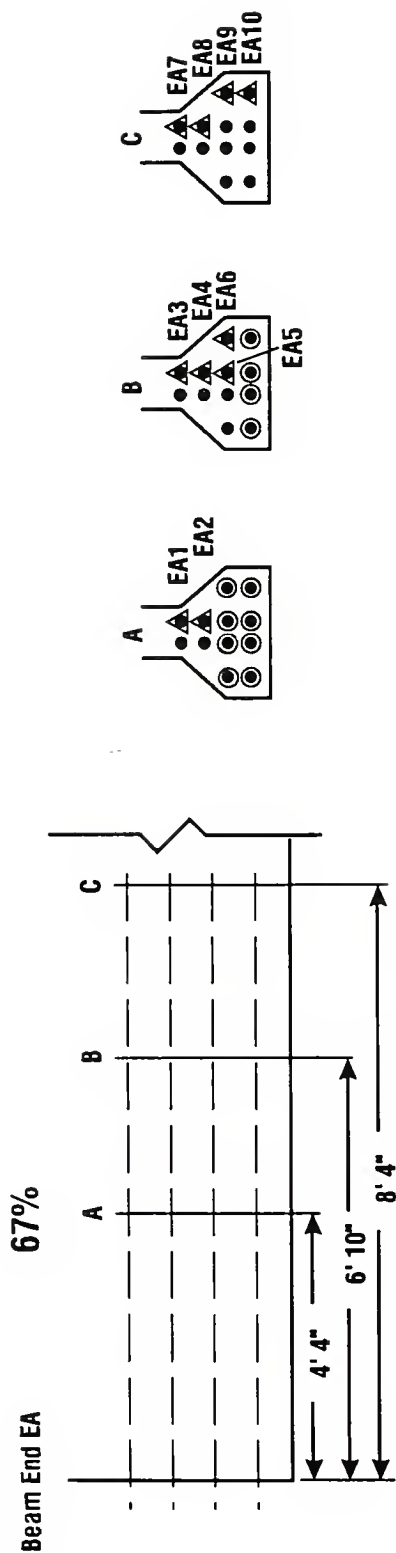
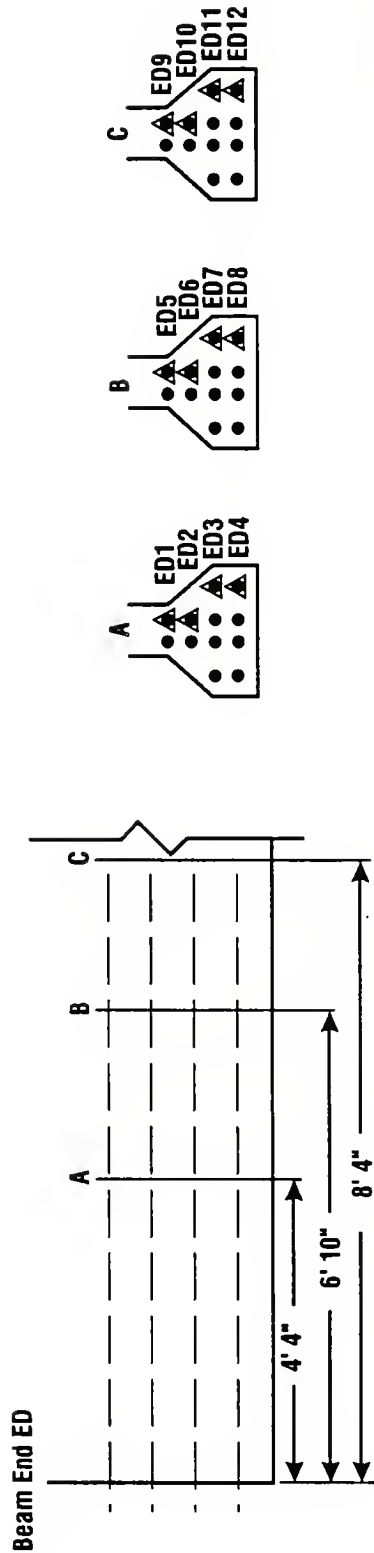


Figure 3.76 Strand Movement, Beam with 50% Debonding
Specimen Set 2

Beam With Debonding



Beam With 0% Debonding



△ Indicates Instrumented Strand ○ Indicates Bond Breaks

Figure (3.77) Strand Debonding Scheme and Instrumentation for Specimen Set 3.

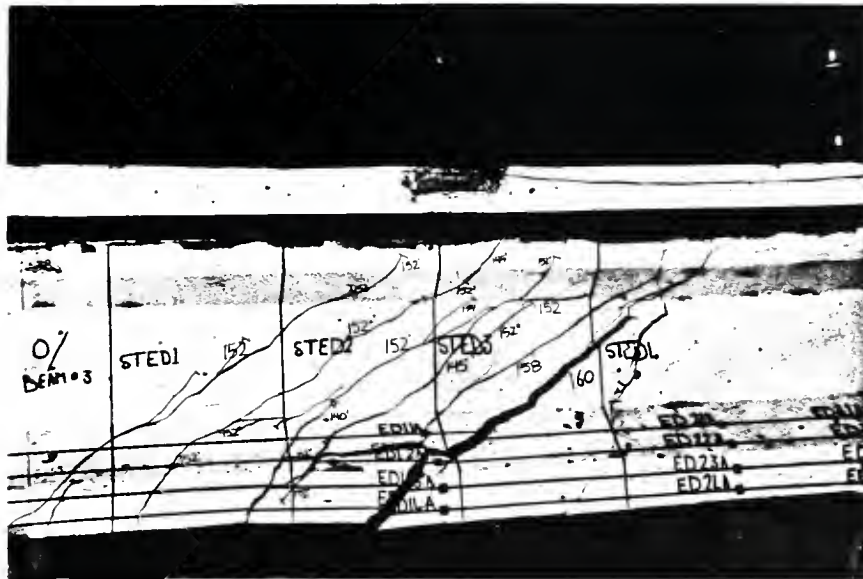


Figure 3.78 Crack Pattern of 0% Debonded Beam at Failure
Specimen Set 3



Figure 3.79 Crack Pattern of 67% Debonded Beam at Failure
Specimen Set 3

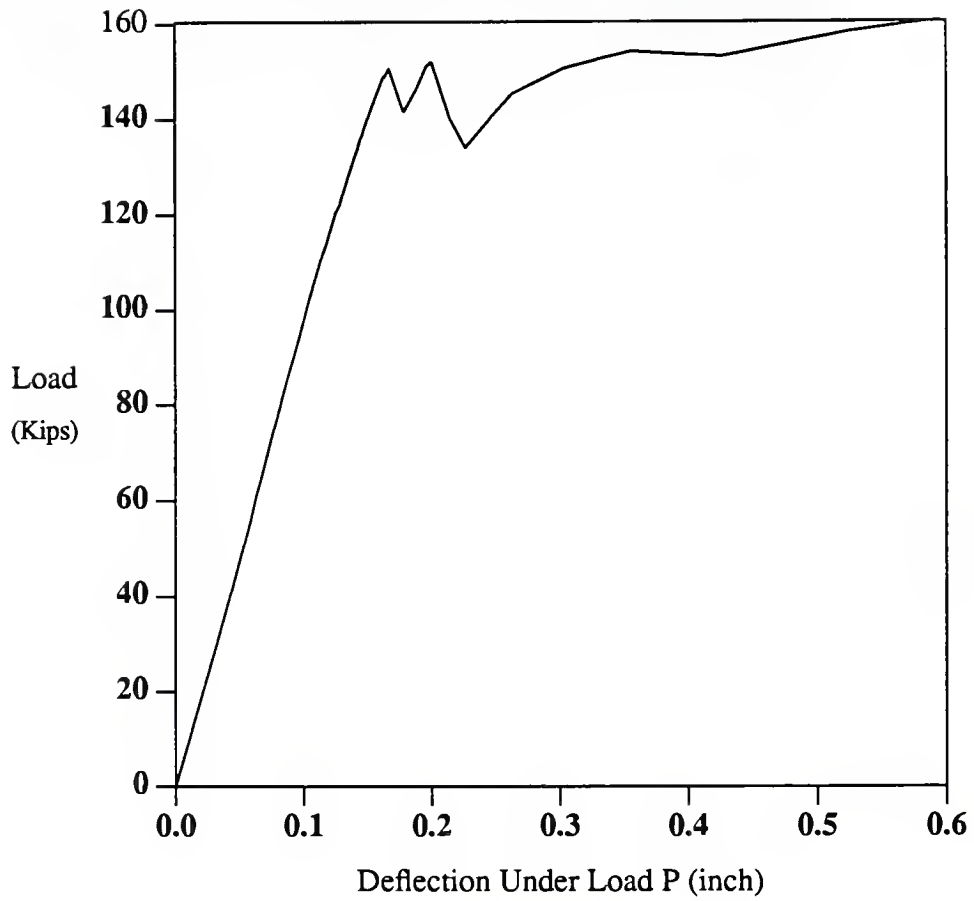


Figure 3.80 Load-Deflection Relationship, Beam with 0% Debonding
(Specimen Set 3)

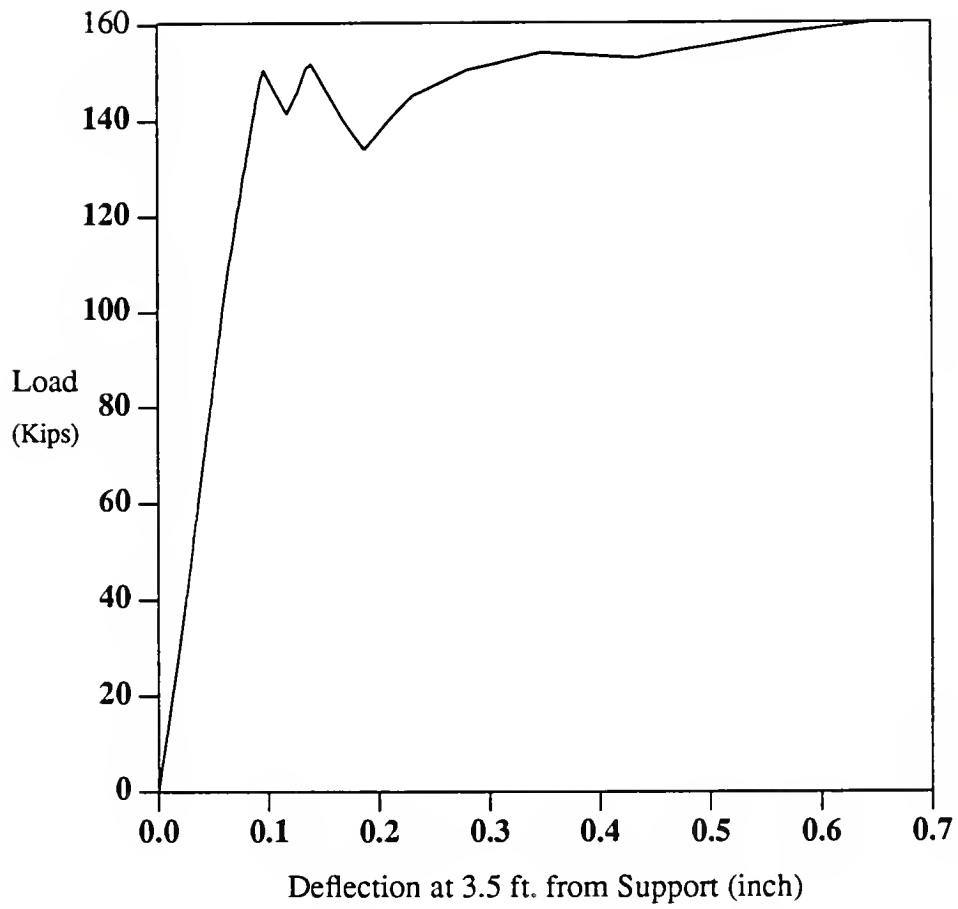


Figure 3.81 Load-Deflection Relationship, Beam with 0% Debonding
(Specimen Set 3)

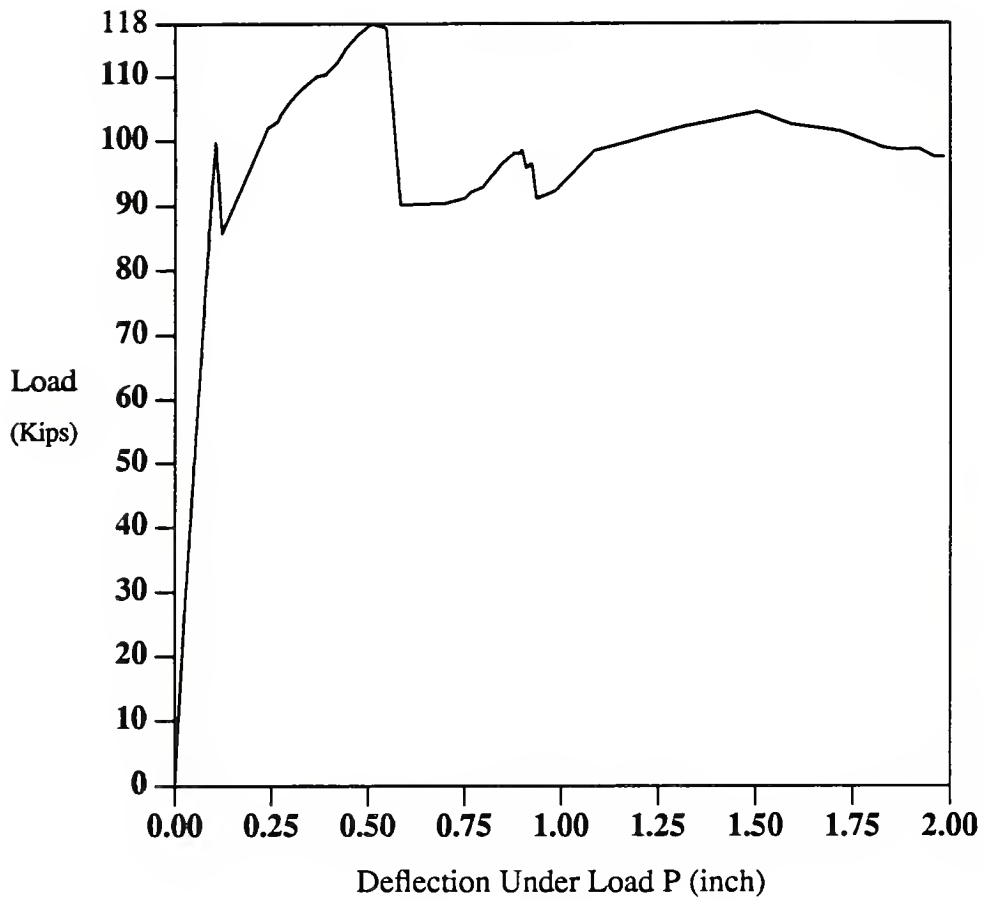


Figure 3.82 Load-Deflection Relationship, for Simply Supported Beam
Beam with 67% Debonding (Specimen Set 3)

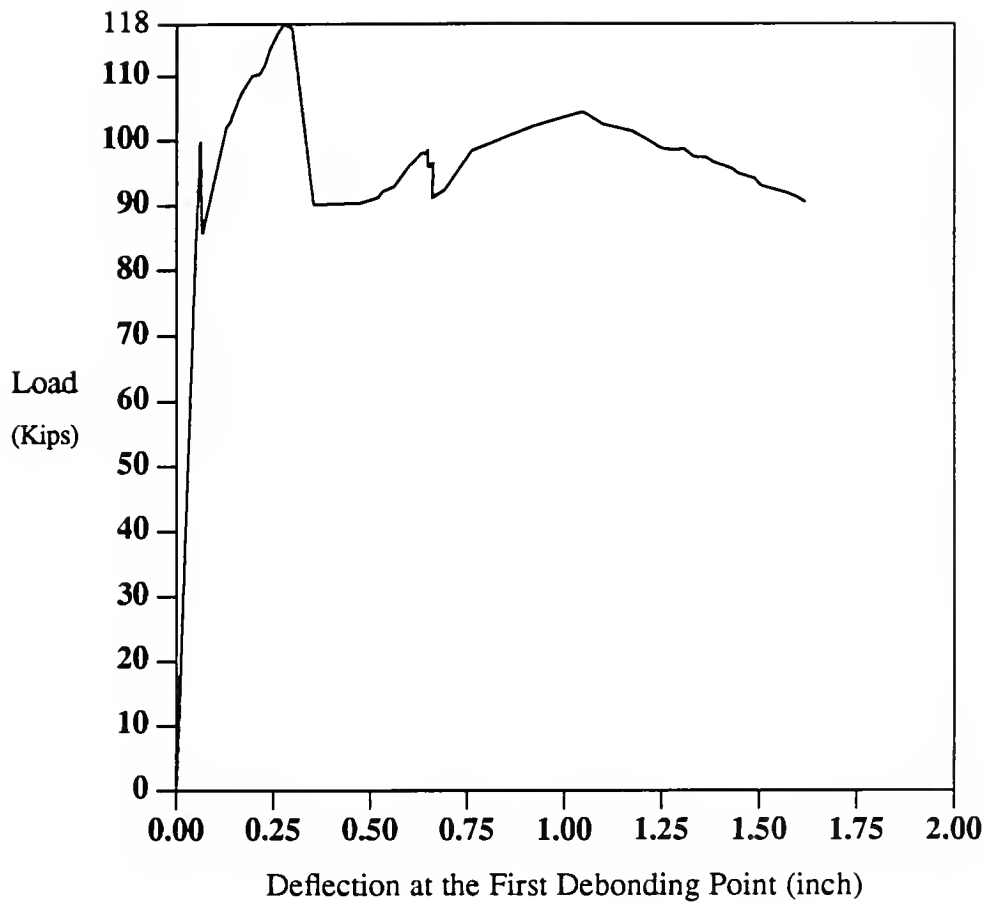


Figure 3.83 Load-Deflection Relationship, for Simply Supported Beam
Beam with 67% Debonding (Specimen Set 3)

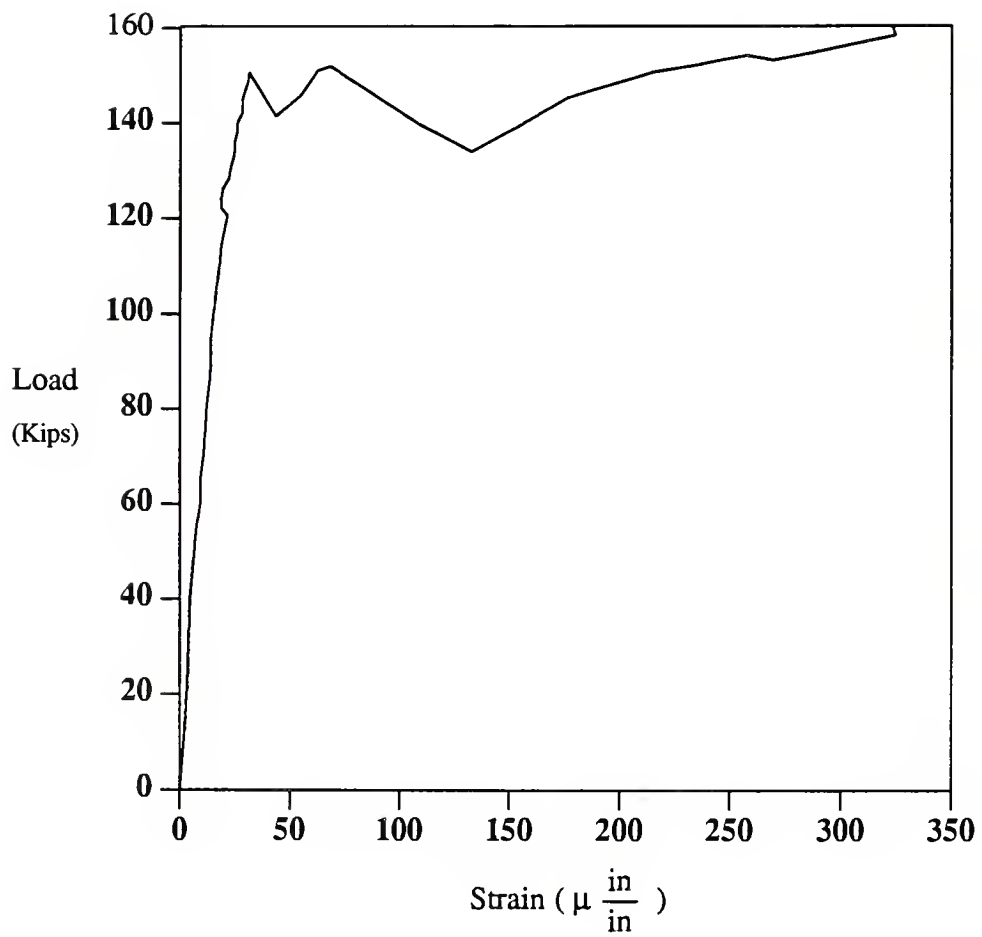


Figure 3.84 Surface Strain in Slab, at 3.5 ft. from Support
Beam with 0% Debonding (Specimen Set 3)

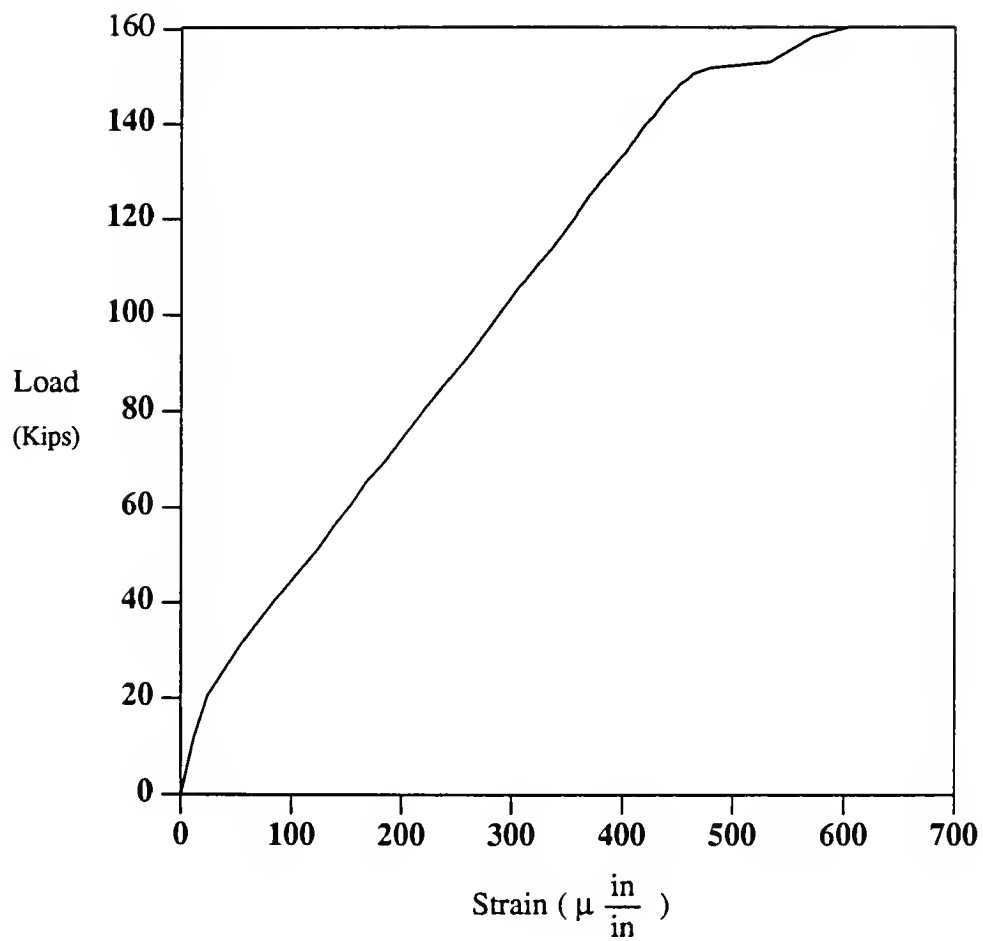


Figure 3.85 Surface Strain in Slab at the Point Load
Beam with 0% Debonding (Specimen Set 3)

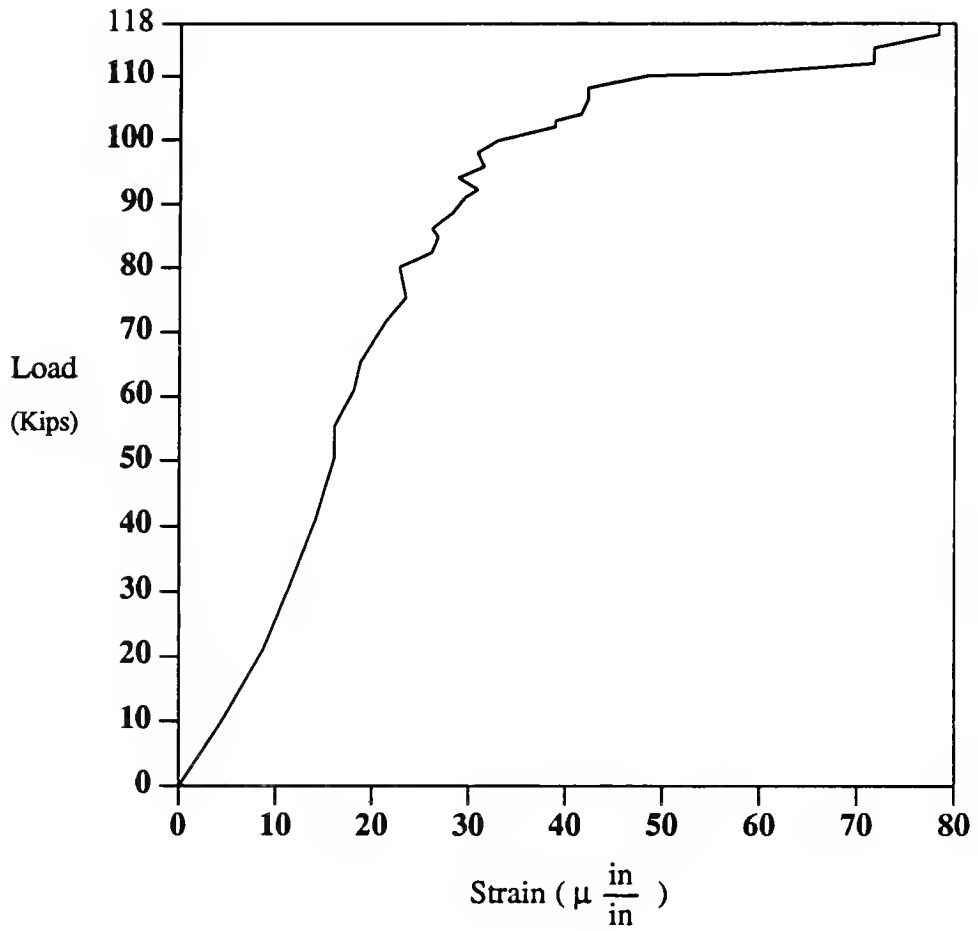


Figure 3.86 Surface Strain in Slab at First Debonding Point
Beam with 67% Debonding (Specimen Set 3)

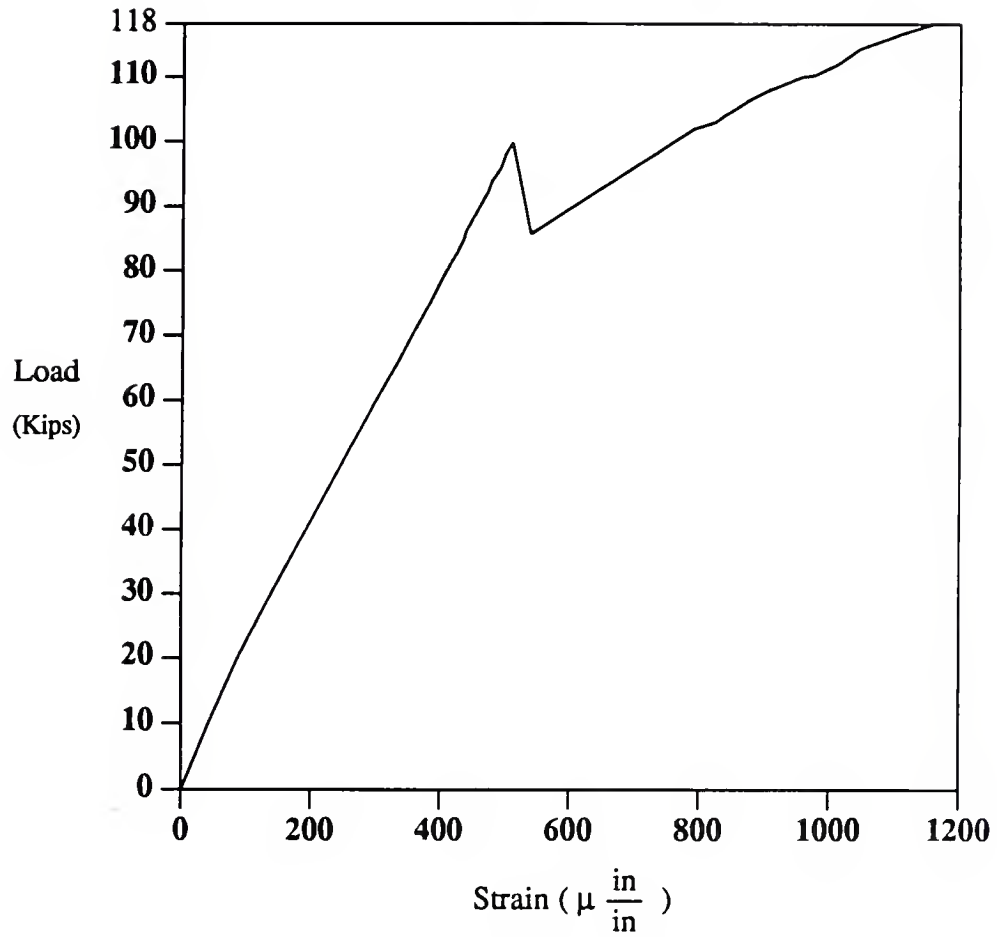


Figure 3.87 Surface Strain in Slab. at the Point Load
Beam with 67% Debonding (Specimen Set 3)

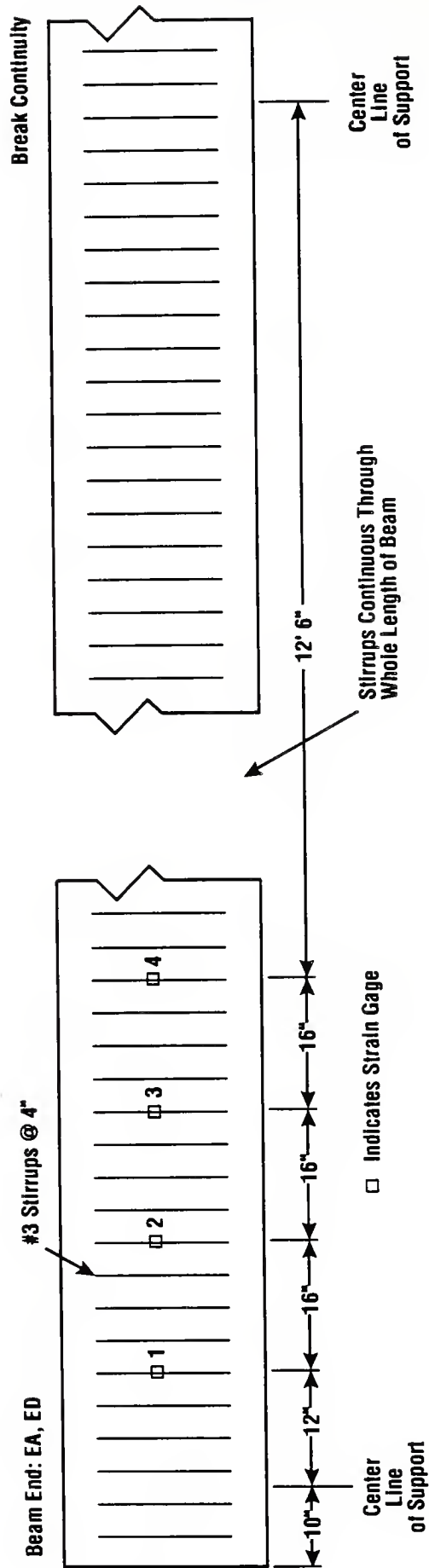


Figure (3.88) Location of Stirrup Reinforcement and Instrumentation for Specimen Sets 3 and 4.

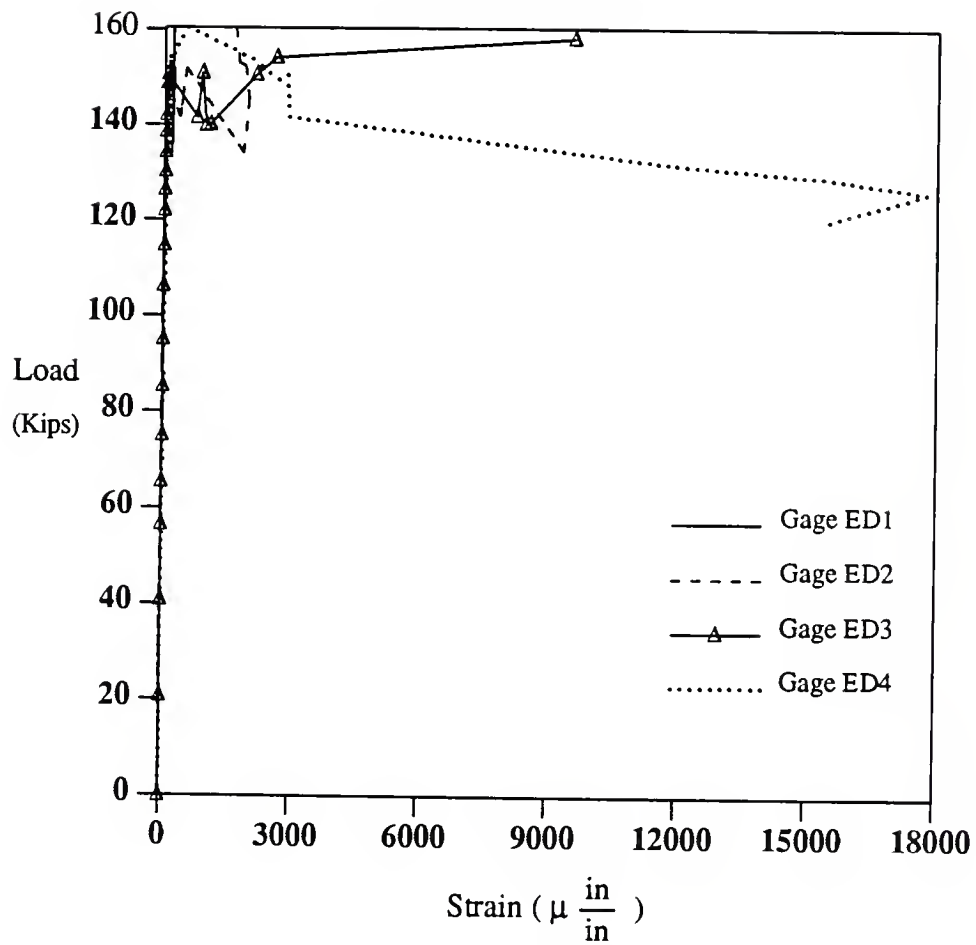


Figure 3.89 Stirrup Strain, Beam with 0% Debonding
(Specimen Set 3)

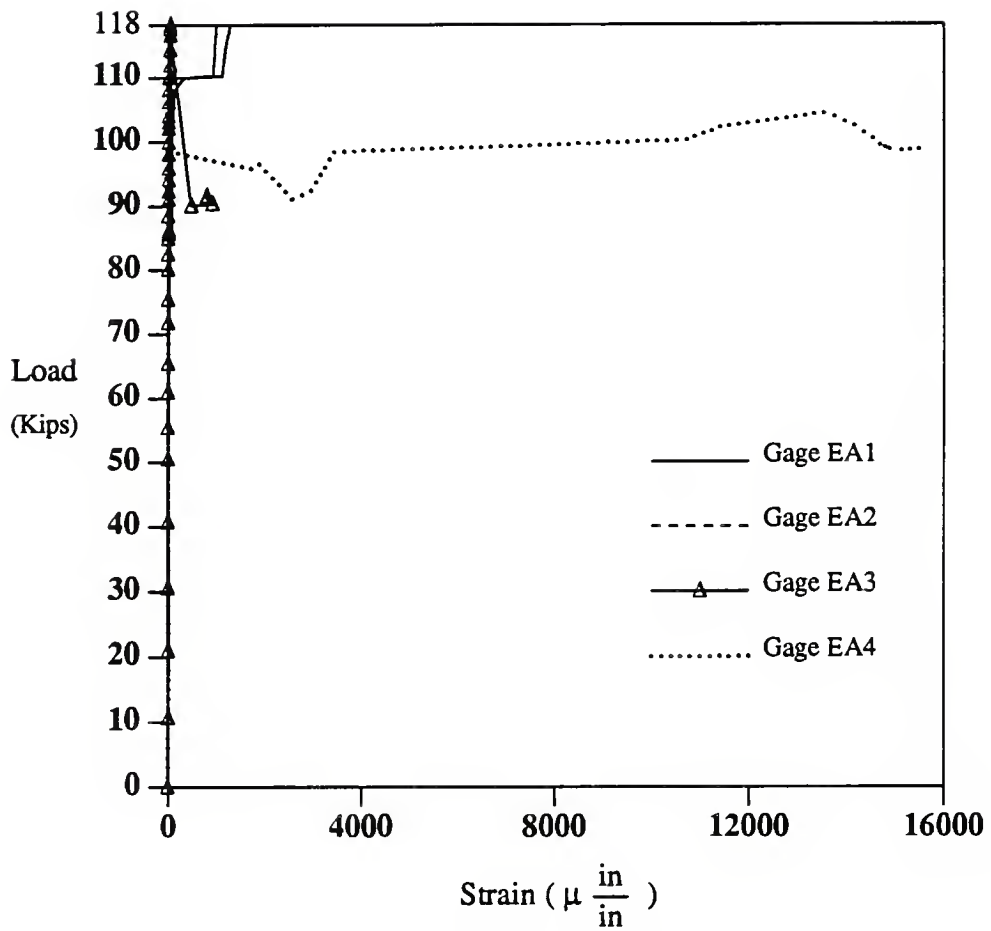


Figure 3.90 Stirrup Strain, Beam with 67% Debonding
Specimen Set 3

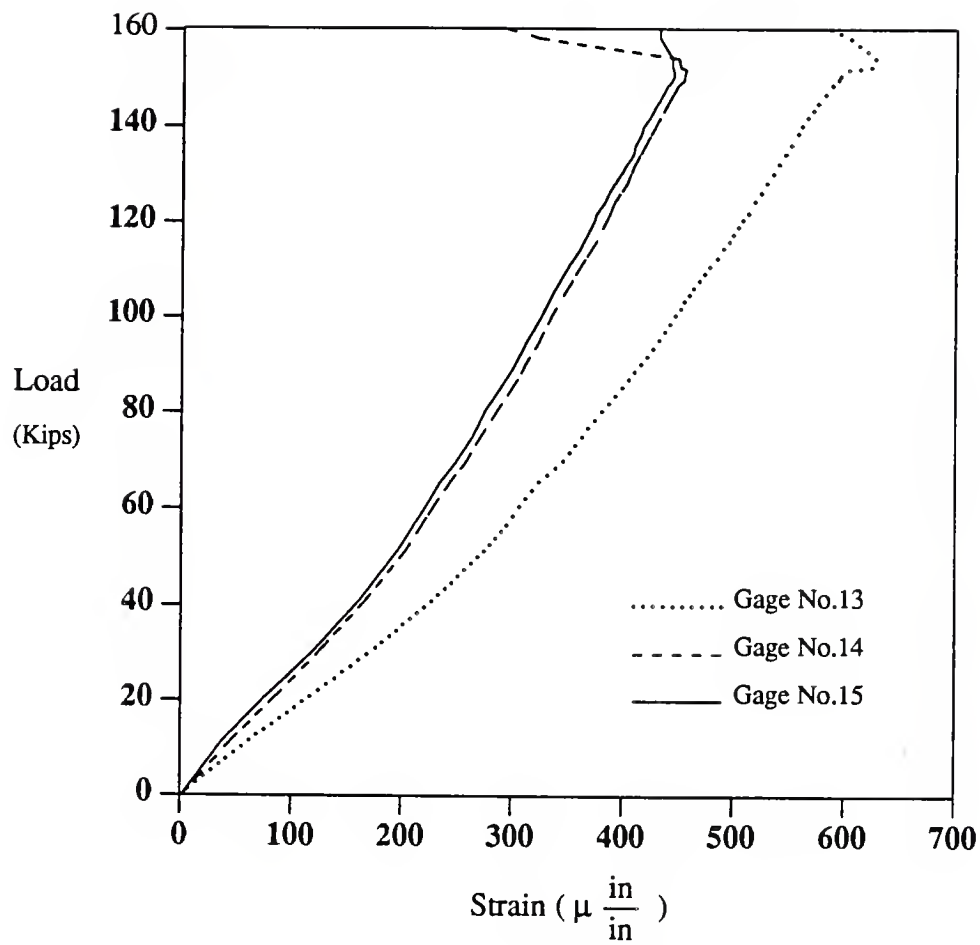


Figure 3.91 Strain in Slab Longitudinal Steel, 6.0 ft. From Support Beam with 0% Debonding (Specimen Set 3)

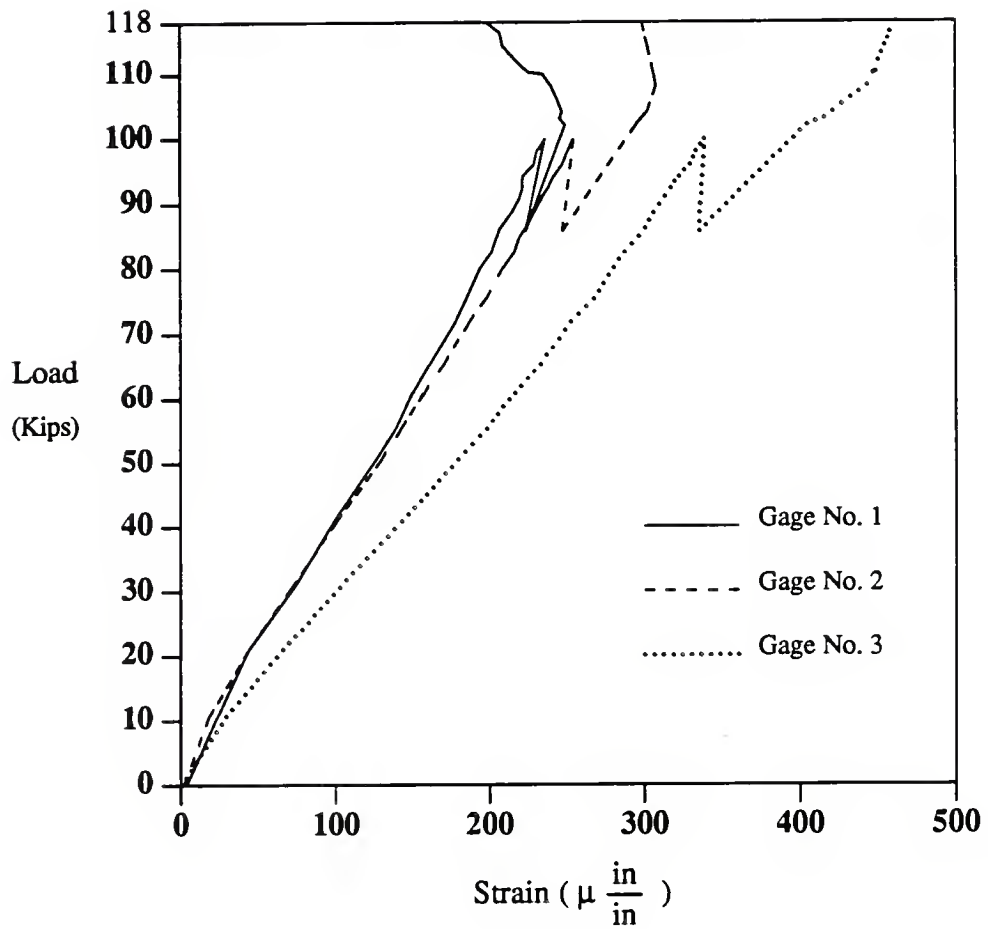


Figure 3.92 Strain in Slab Longitudinal Steel, at 6.0 ft. from Support Beam with 67% Debonding (Specimen Set 3)

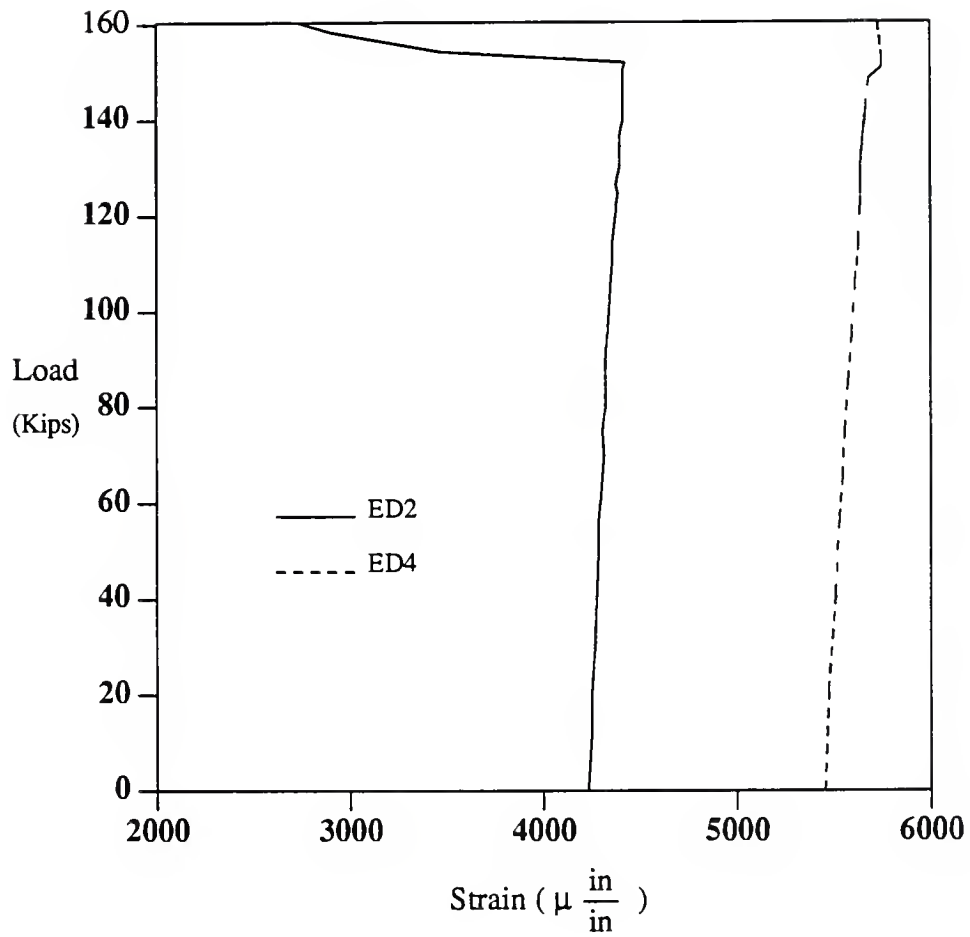


Figure 3.93 Strand Strains, at 3.5 ft. from Support
Beam with 0% Debonding (Specimen Set 3)

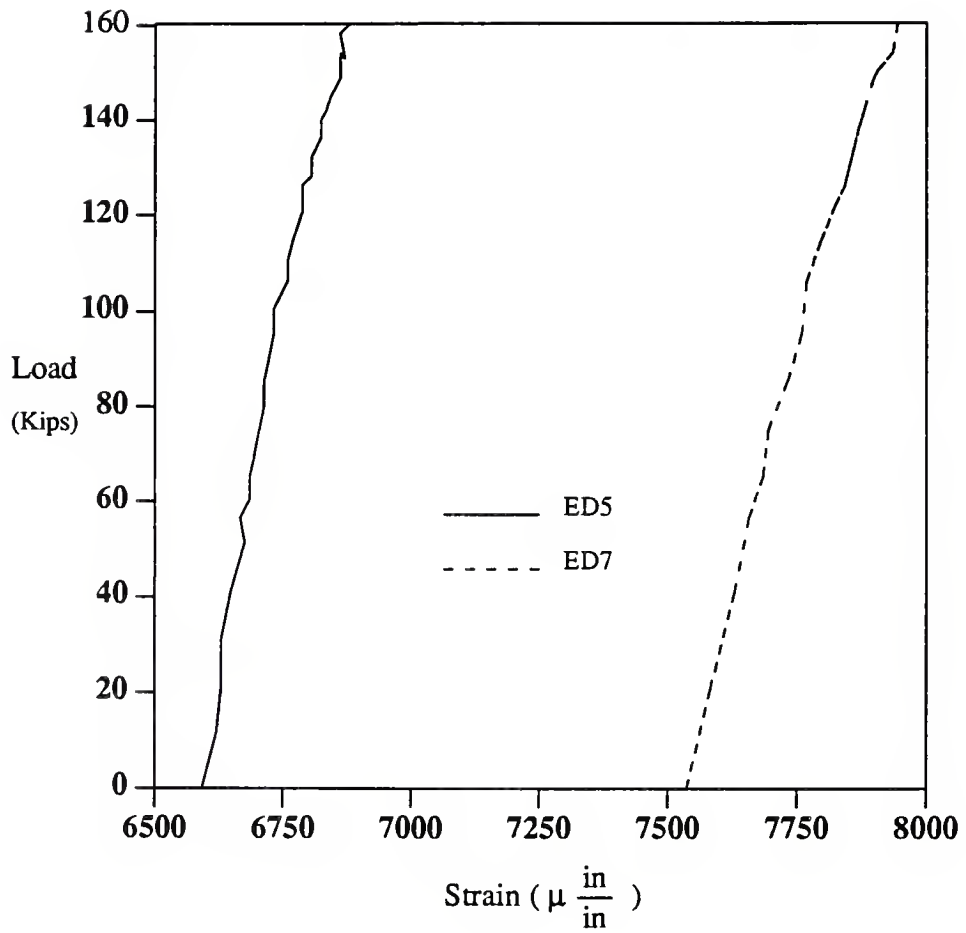


Figure 3.94 Strand Strains, at 6.0 ft. from Support
Beam with 0% Debonding (Specimen Set 3)

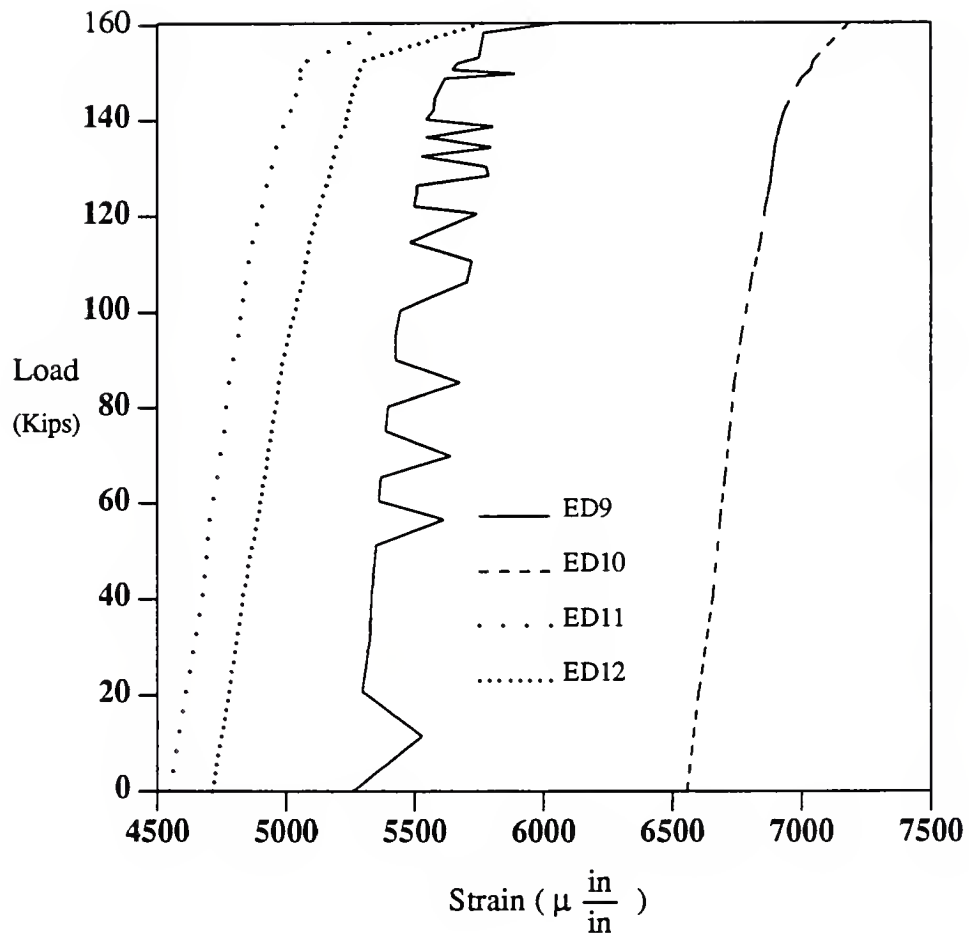


Figure 3.95 Strand Strains at the point Load Beam with 0% Debonding (Specimen Set 3)

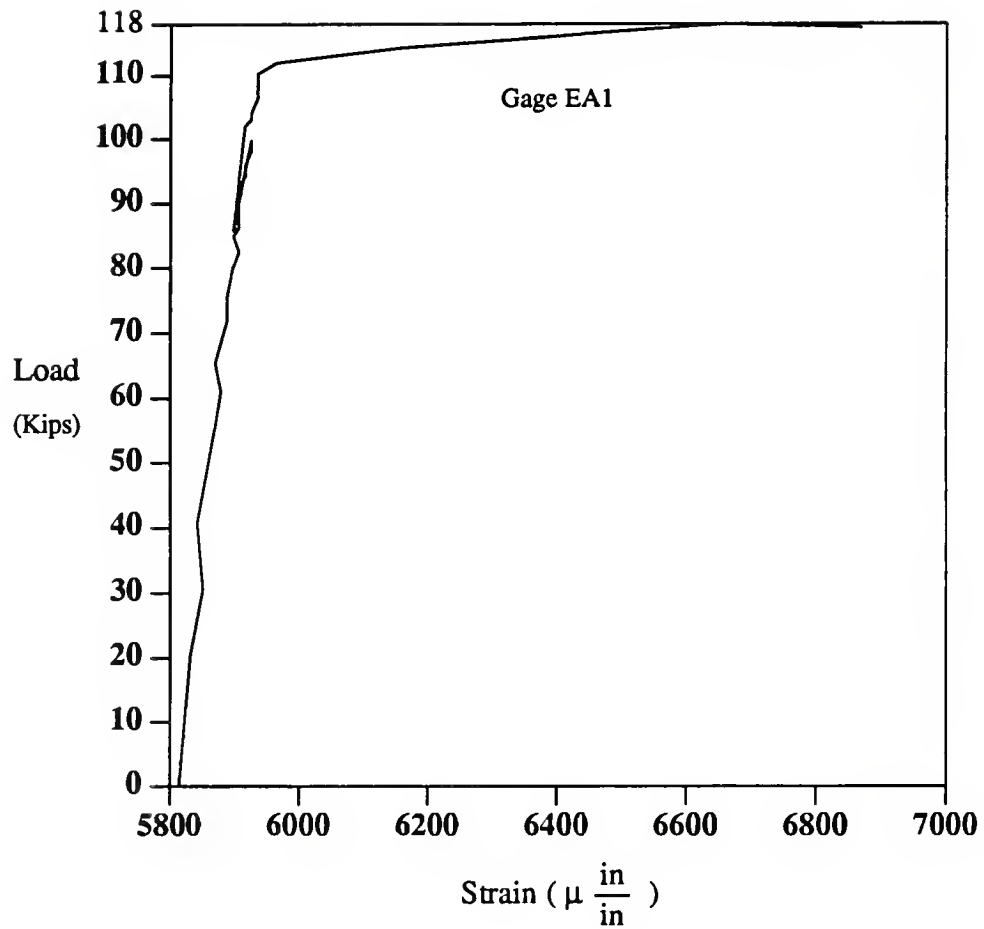


Figure 3.96 Strand Strain at First Debonding Point
Beam with 67% Debonding (Specimen Set 3)

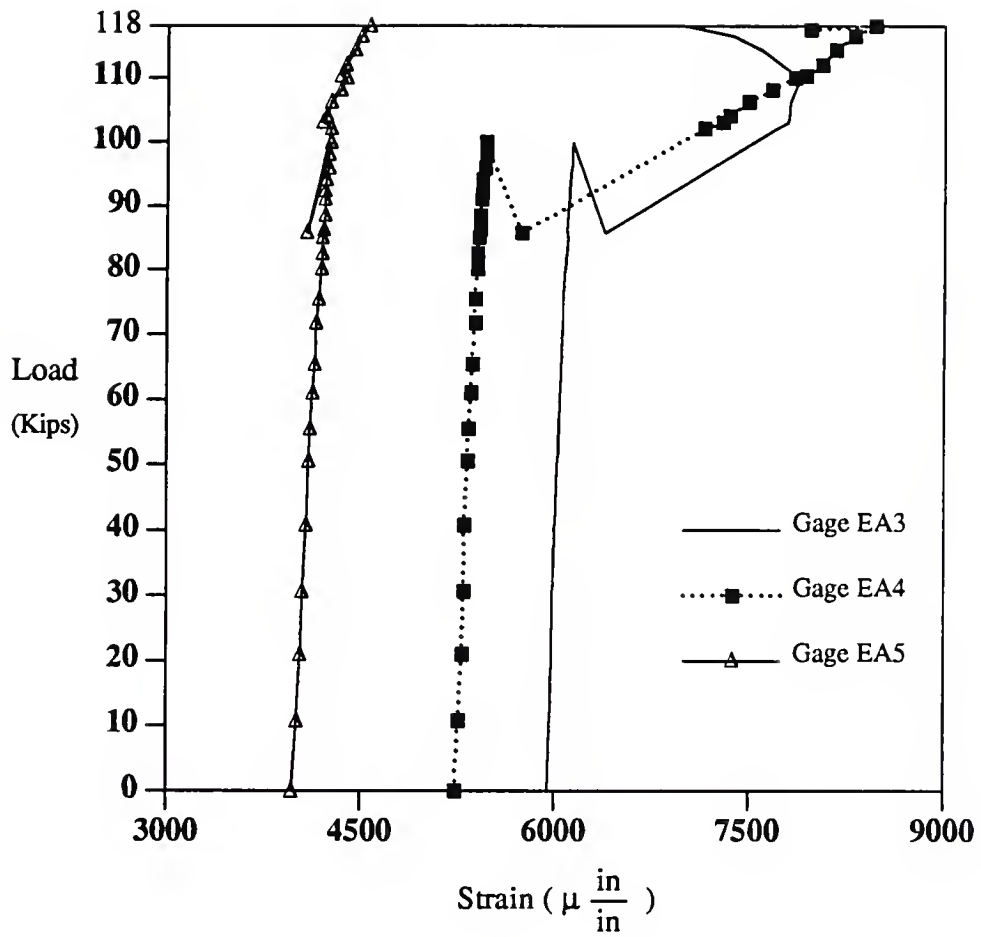


Figure 3.97 Strand Strains at Second Debonding Point
Beam with 67% Debonding (Specimen Set 3)

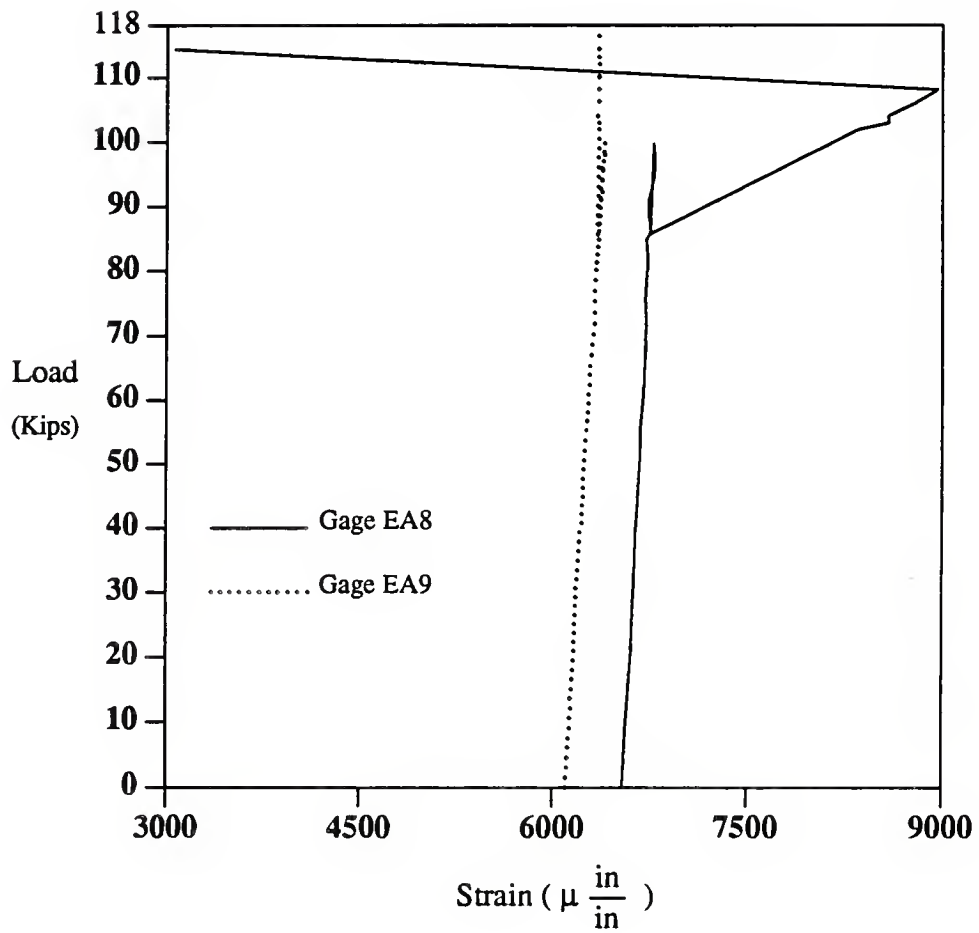


Figure 3.98 Strand Strains at at the Point Load
Beam with 67% Debonding (Specimen Set 3)

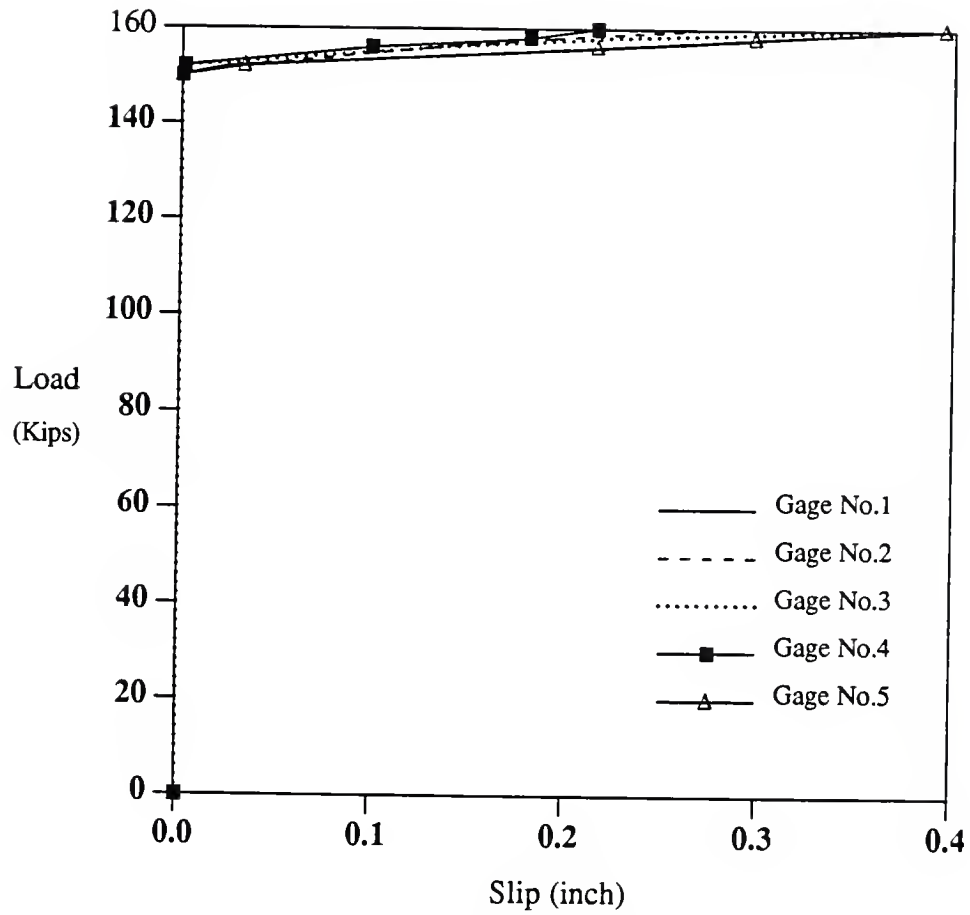


Figure 3.99 Strand Slip, Beam with 0% Debonding
Specimen Set 3

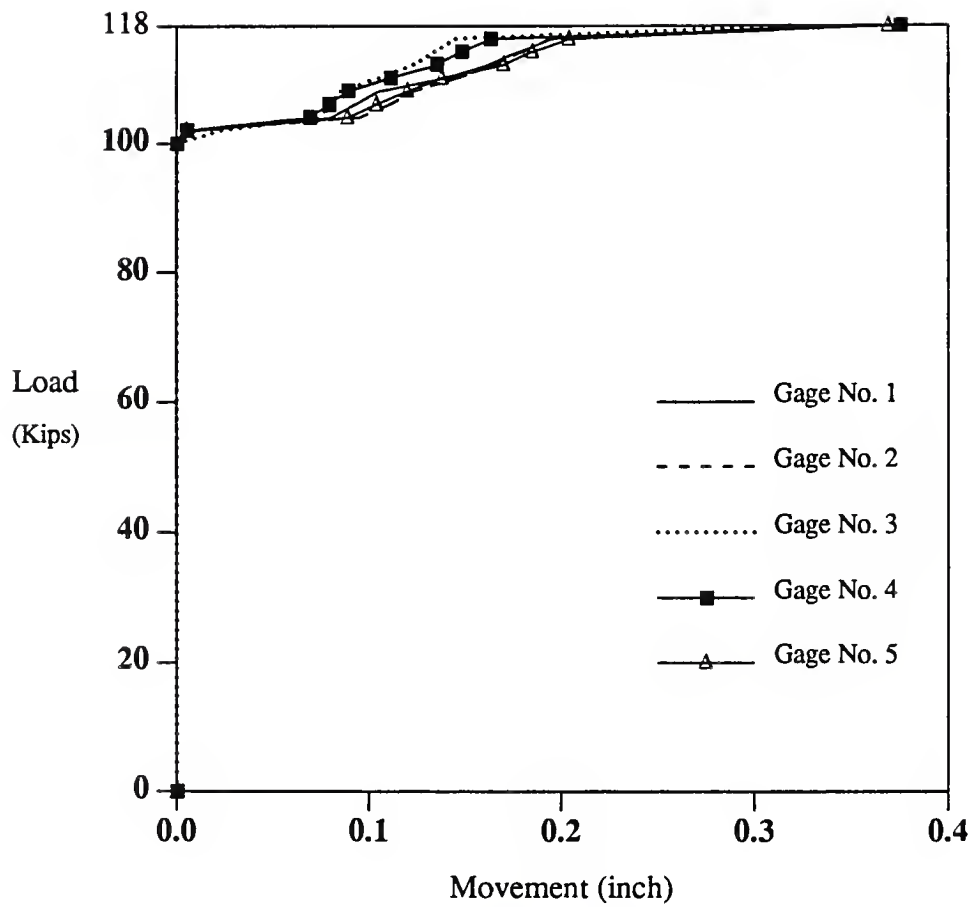
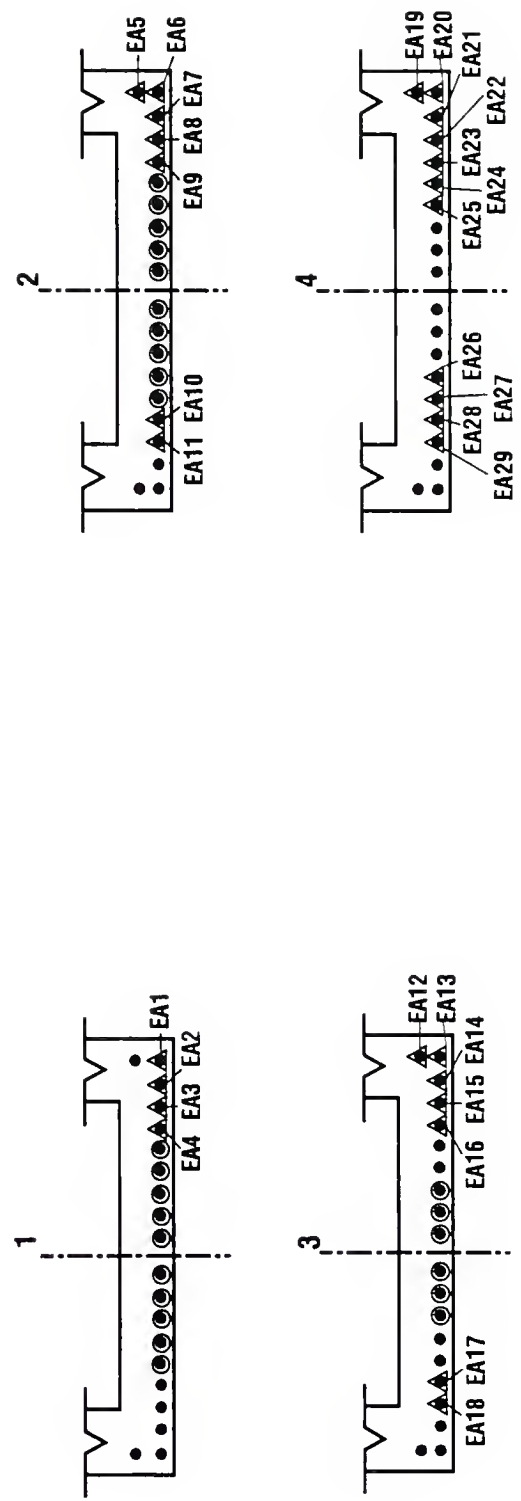
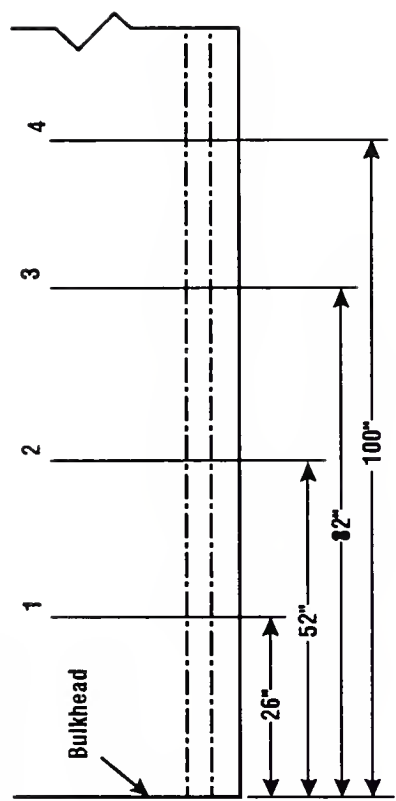


Figure 3.100 Strand Movement, Beam with 67% Debonding
Specimen Set 3

Beam With Debonding

Beam End EA
50 % Debonding

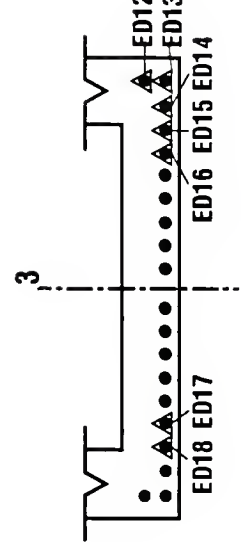
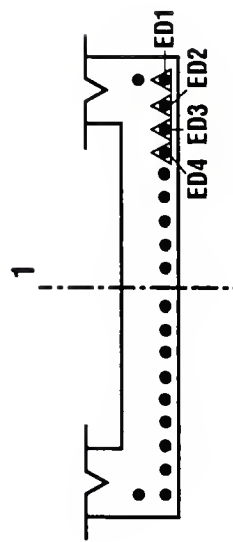
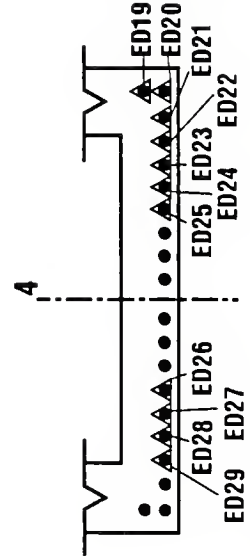
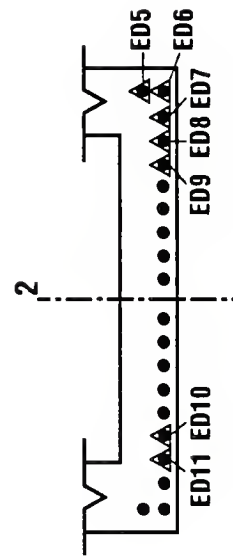
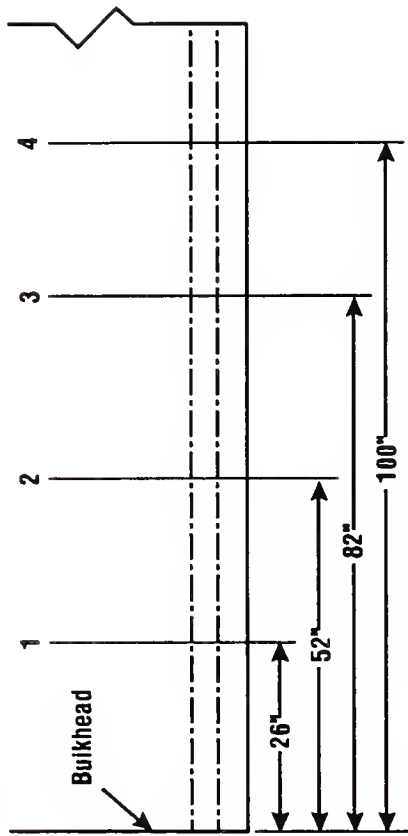


△ Indicates Instrumented Strand ○ Indicates Bond Breaks

Figure (3.101) Strand Debonding Scheme and Instrumentation for Specimen Set 4.

Beam With 0% Debonding

Beam End ED
0 % Debonding



○ Indicates Bond Breaks △ Indicates Instrumented Strand

Figure (3.10I, continued) Strand Debonding Scheme and Instrumentation for Specimen Set 4.



Figure 3.102 Crack Pattern of 0% Debonded Beam at Failure
Specimen Set 4

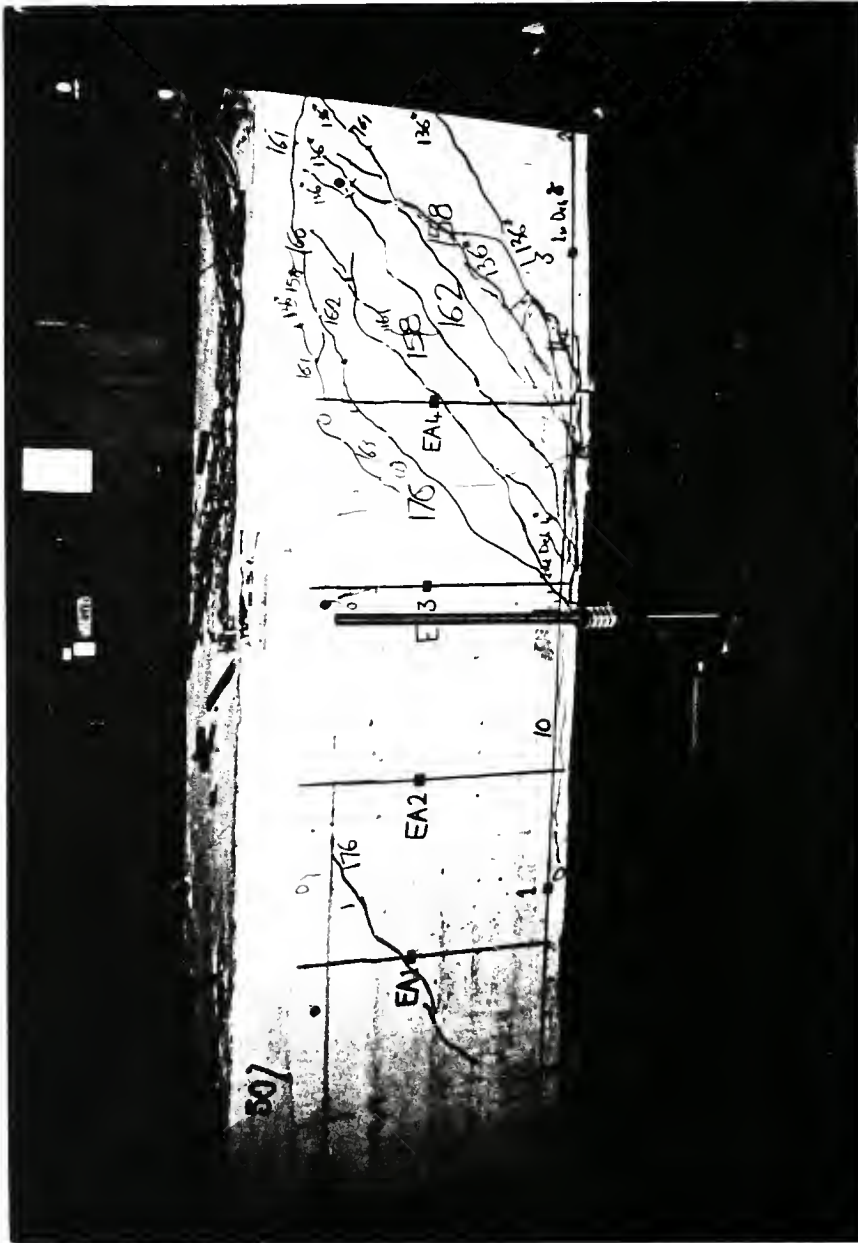


Figure 3.103 Crack Pattern of 50% Debonded Beam at Failure
Specimen Set 4

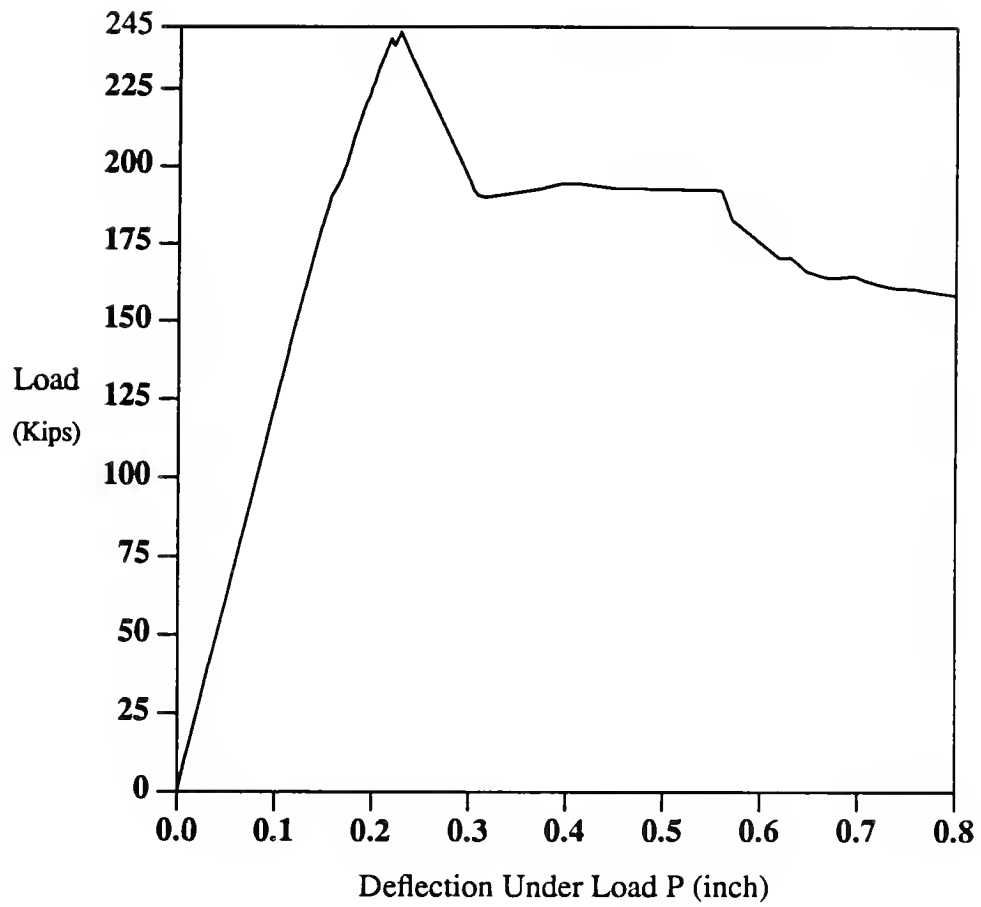


Figure 3.104 Load Deflection Relationship, Beam with 0% Debonding
(Specimen Set 4)

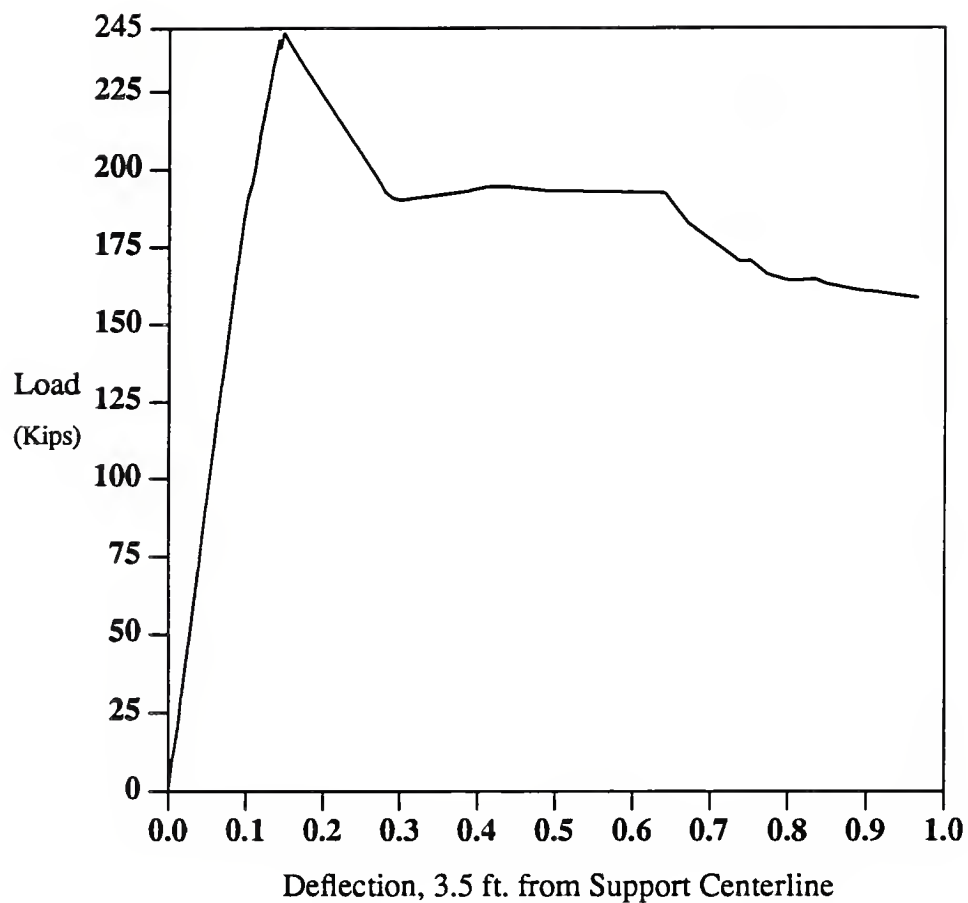


Figure 3.105 Load Deflection Relationship, Beam with 0% Debonding
(Specimen Set 4)

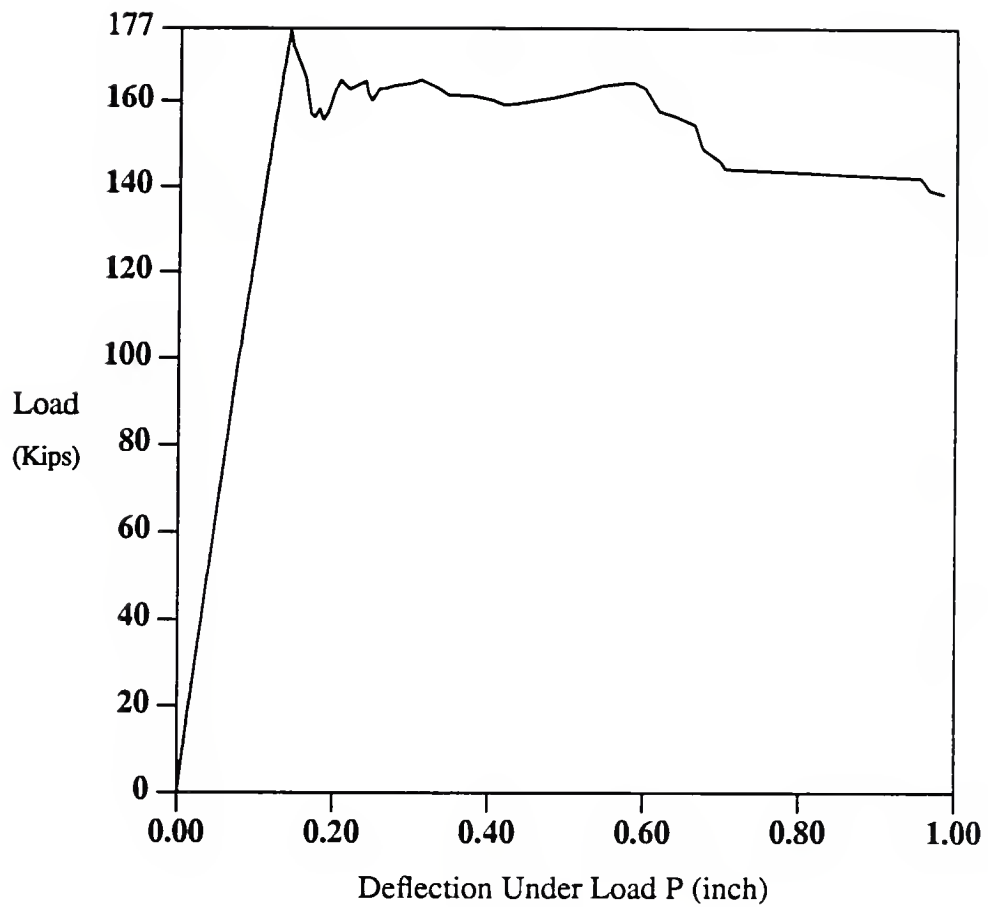


Figure 3.106 Load-Deflection Relationship, Beam with 50% Debonding
(Specimen Set 4)

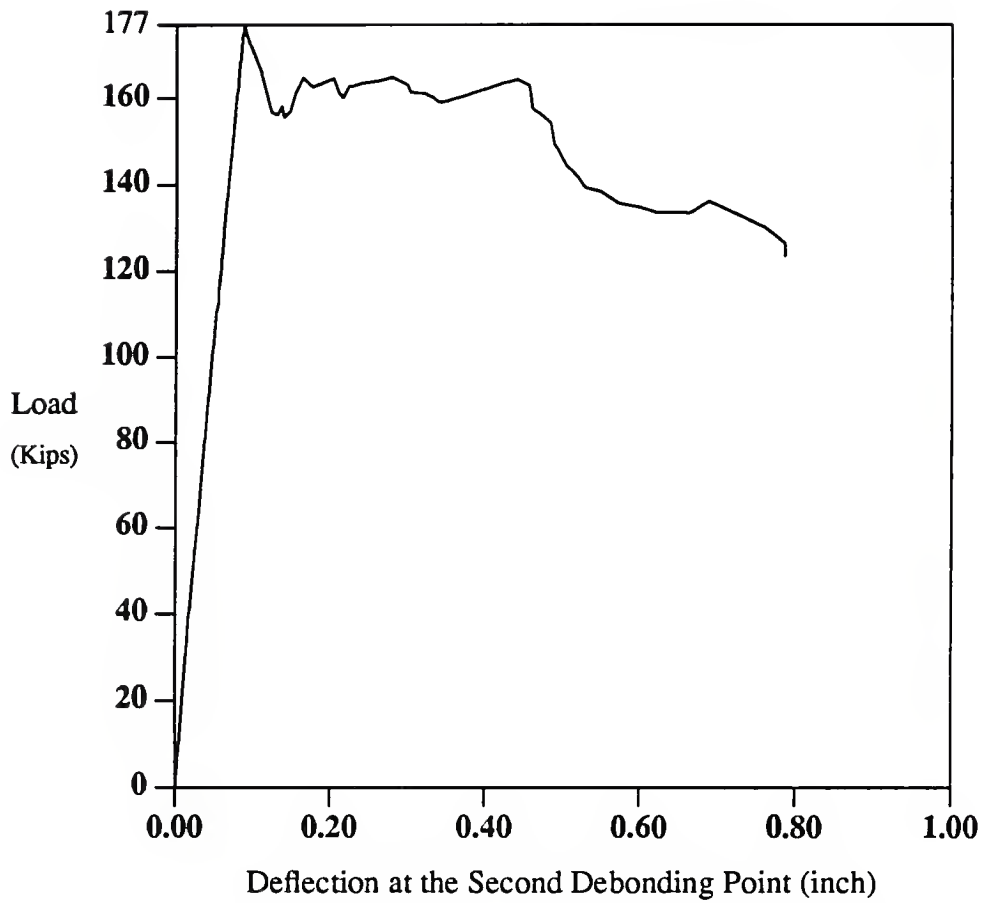


Figure 3.107 Load Deflection Relationship, Beam with 50% Debonding
(Specimen Set 4)

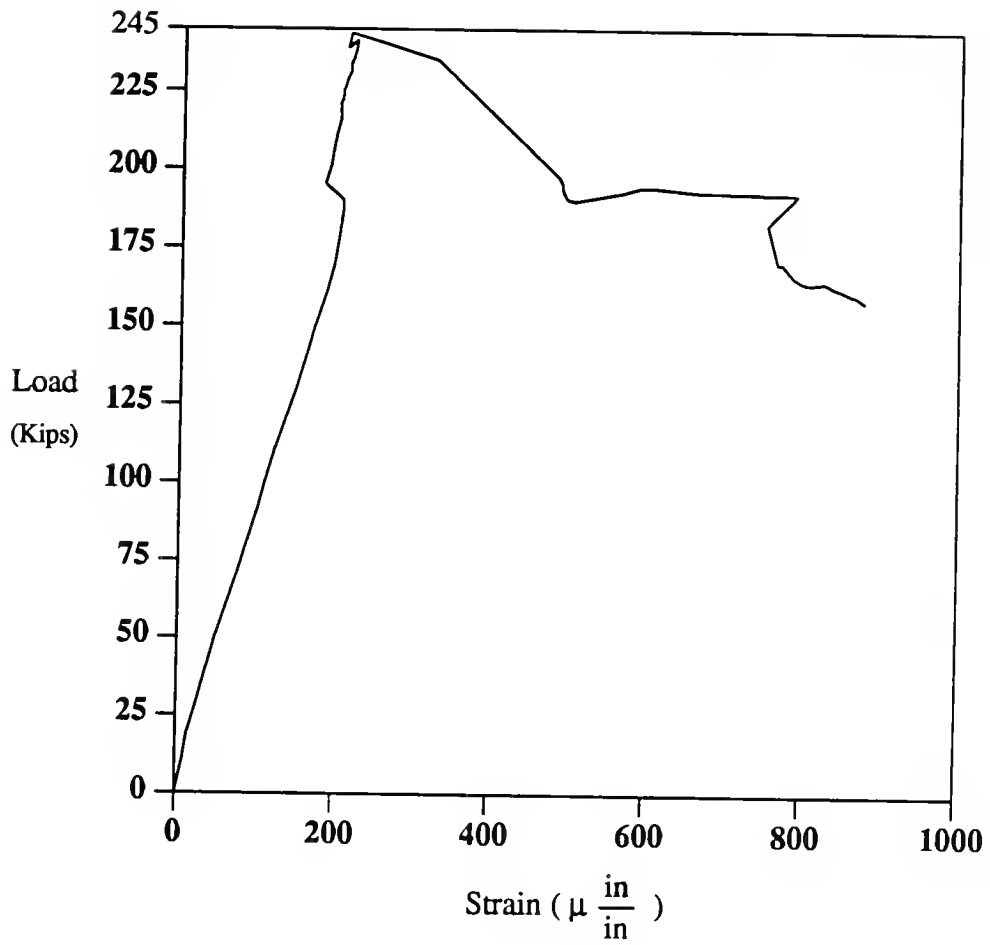


Figure 3.108 Surface Strain in Slab, 3.5 ft. from Support Centerline
Beam with 0% Debonding (Specimen Set 4)

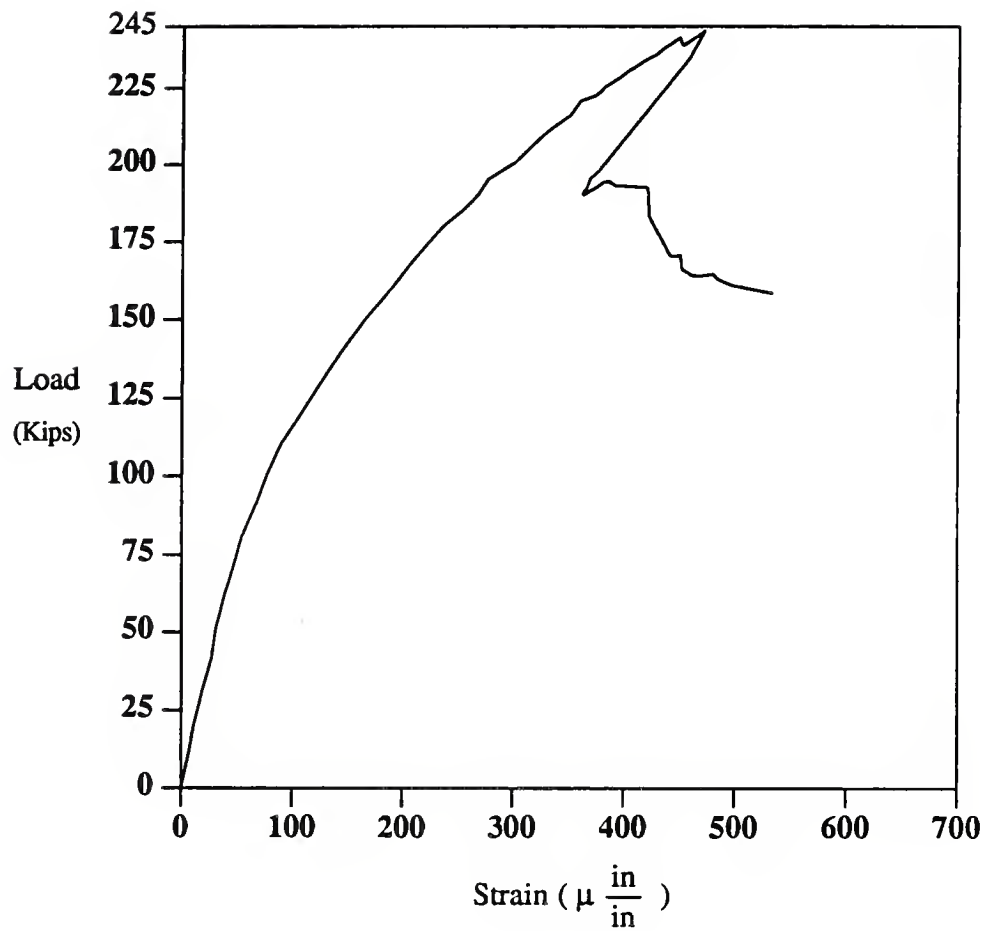


Figure 3.109 Surface Strain in Slab, at the Point Load Beam with 0% Debonding (Specimen Set 4)

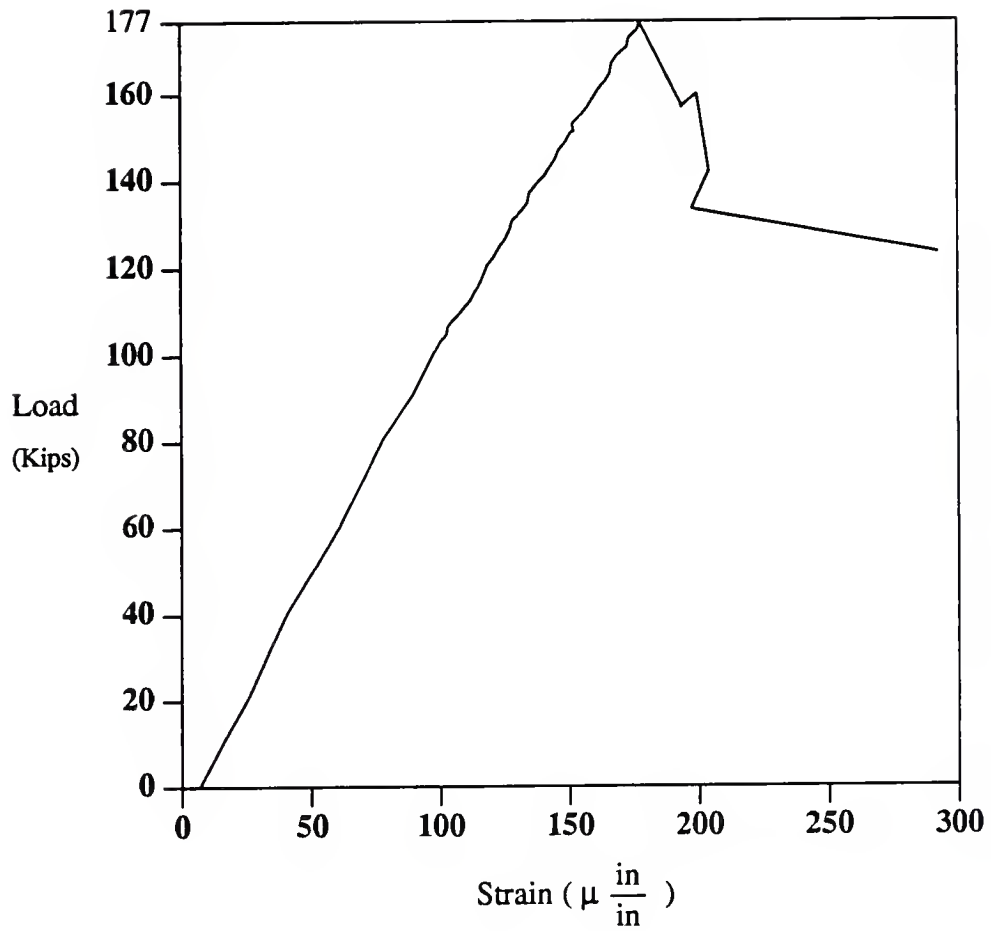


Figure 3.110 Surface Strain in Slab at First Debonding Point
Beam with 50% Debonding (Specimen Set 4)

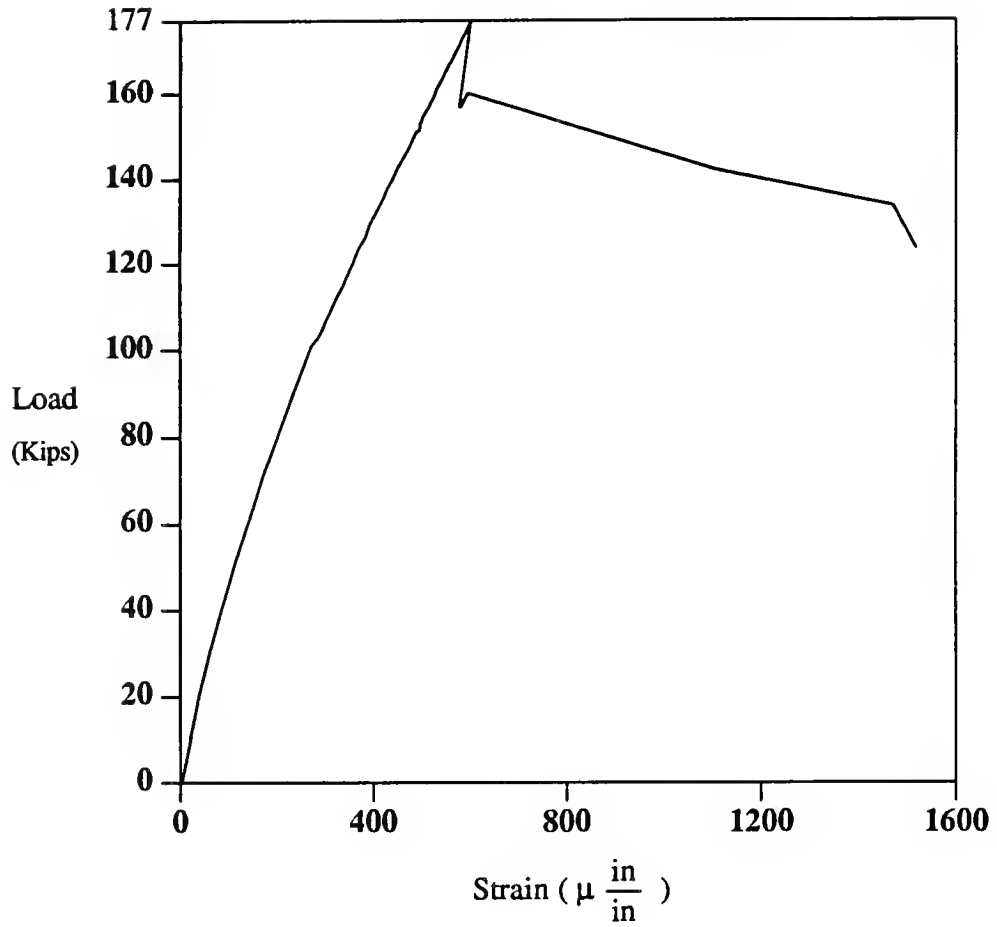


Figure 3.111 Surface Strain in Slab at the Point Load
Beam with 50% Debonding (Specimen Set 4)

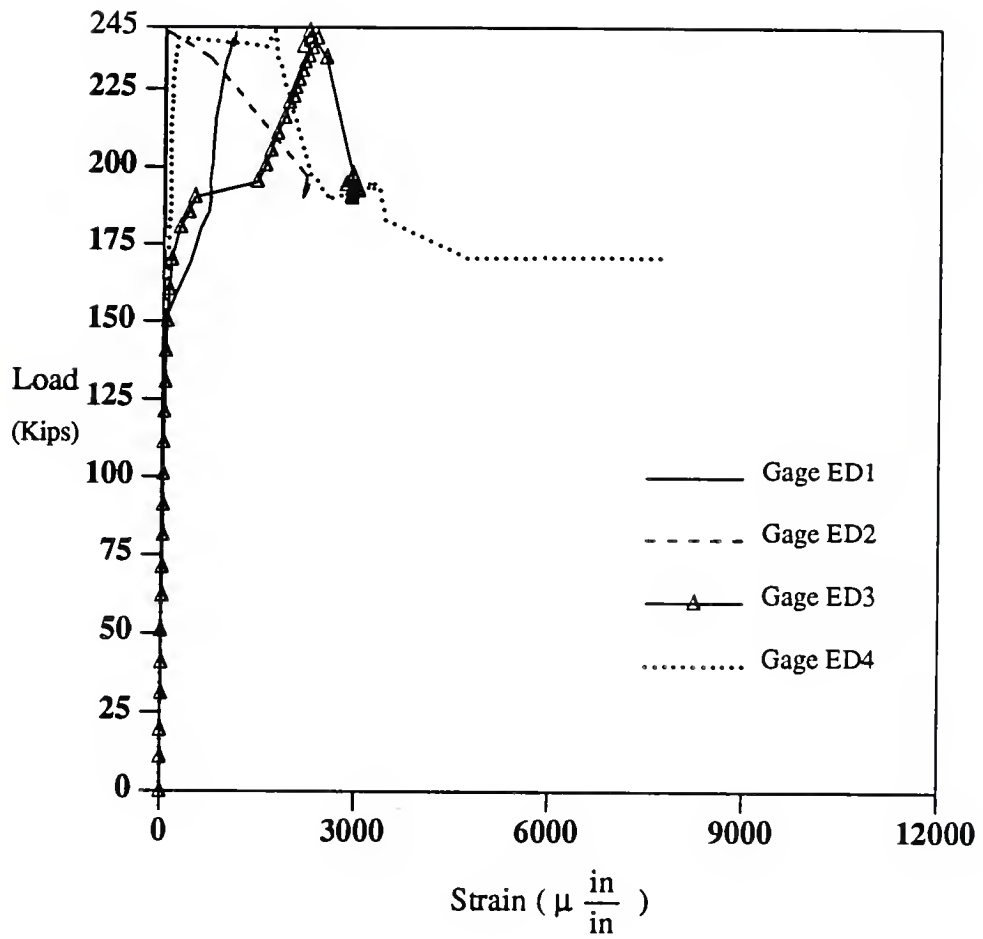


Figure 3.112 Stirrup Strain, Beam with 0% Debonding
Specimen Set 4

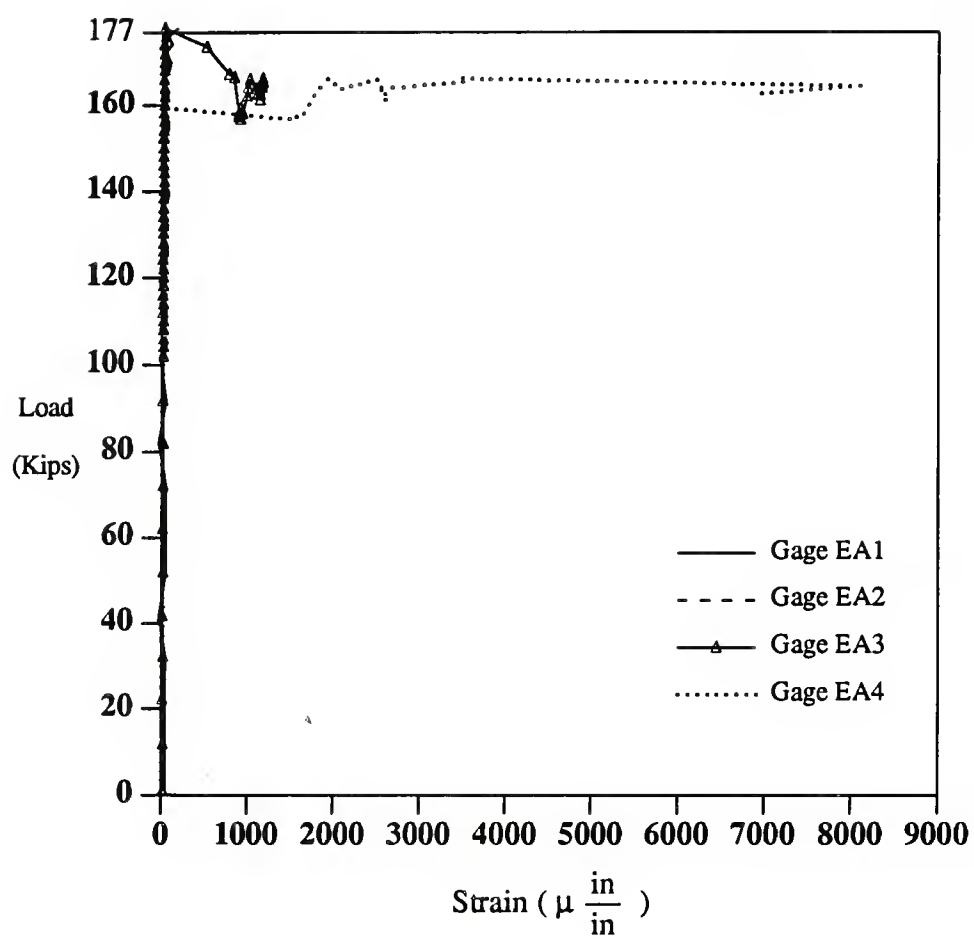
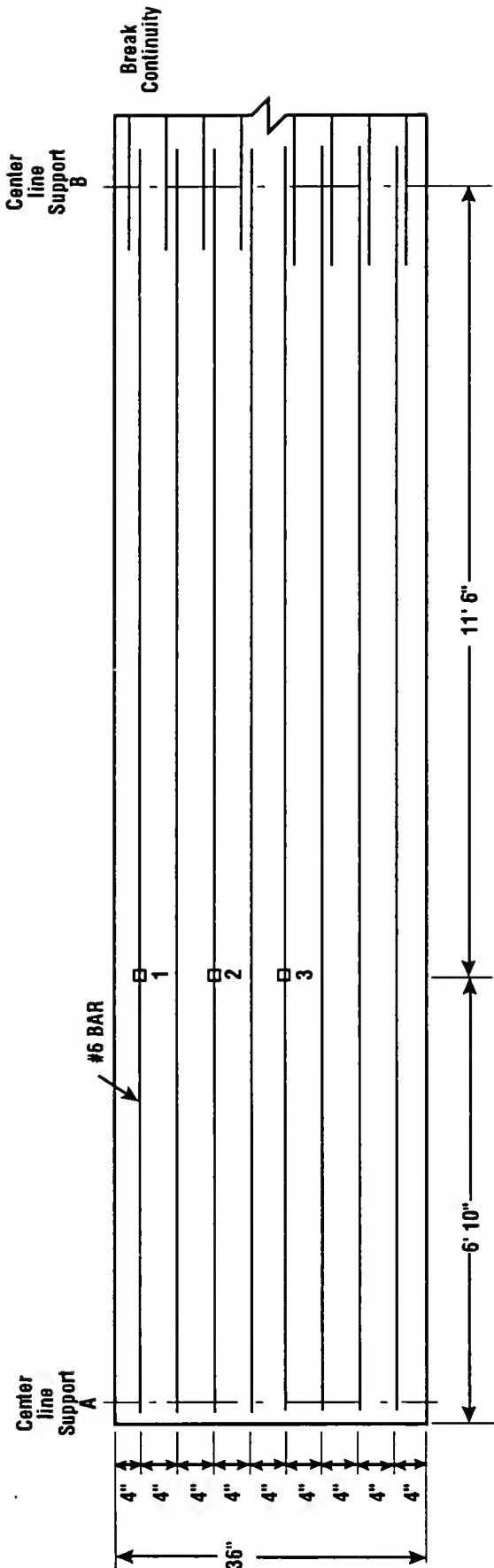


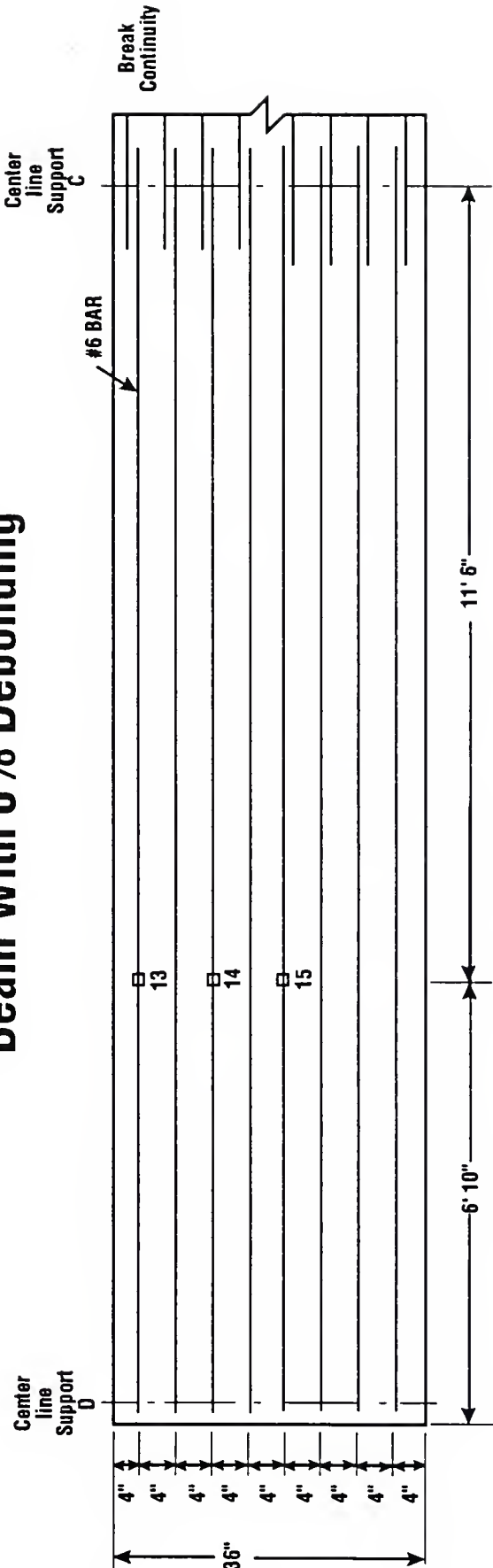
Figure 3.113 Stirrup Strain, Beam with 50% Debonding
Specimen Set 4

Beam With 50% Debonding



- 180 -

Beam With 0% Debonding



□ Indicates Strain Gage Location

Figure 3.114 Slab Longitudinal Steel Instrumentation for Specimen Set 4

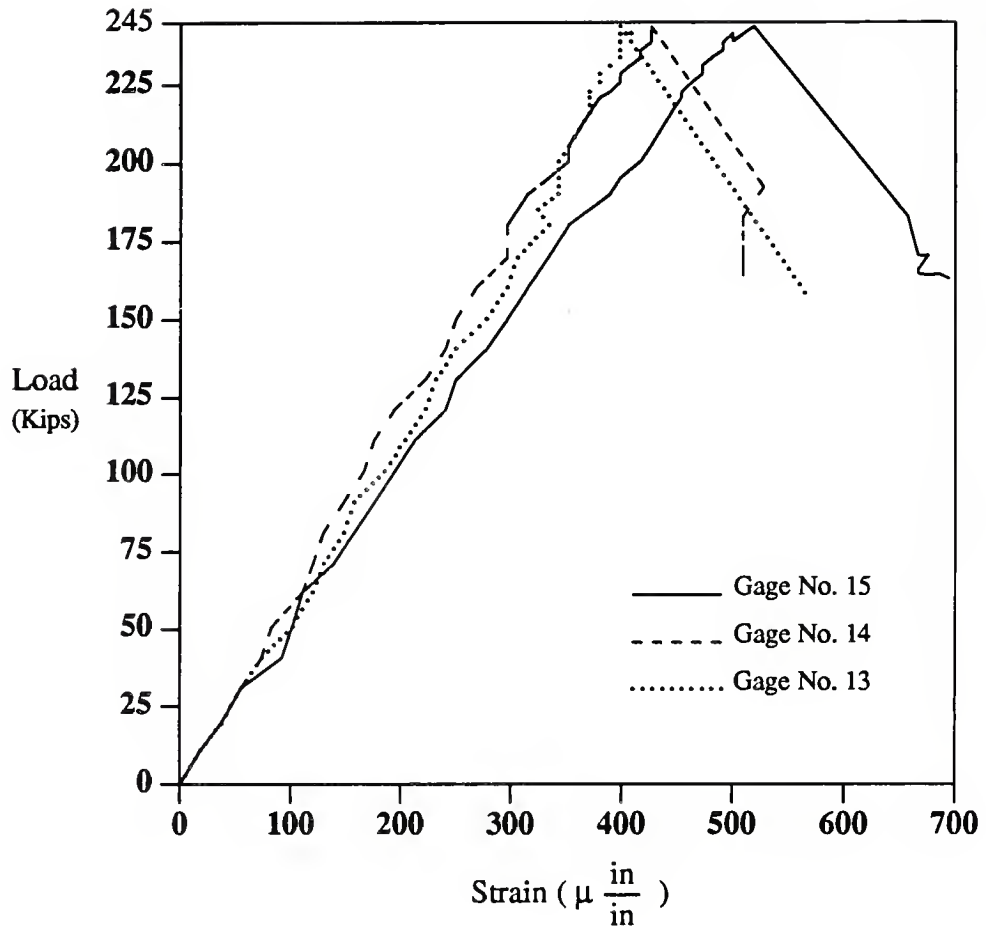


Figure 3.115 Strain in Slab Longitudinal Steel, at 6.0 ft. From Support Beam with 0% Debonding (Specimen Set 4)

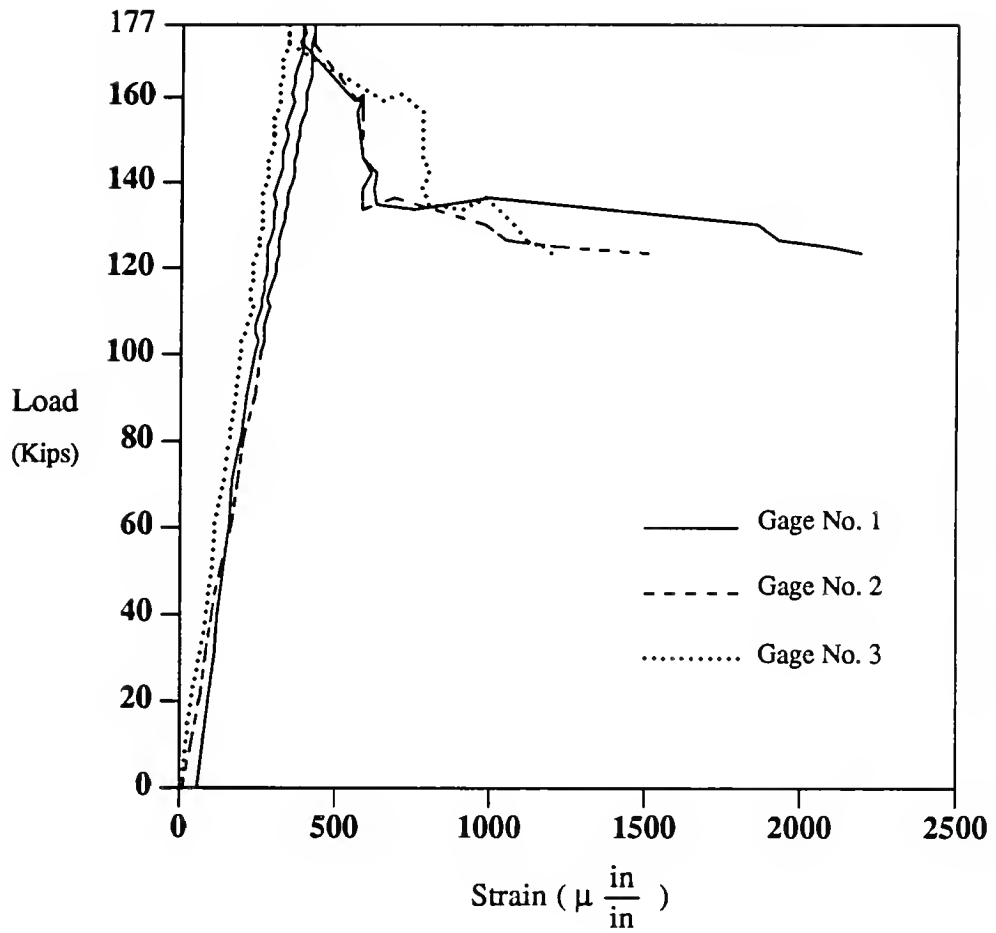


Figure 3.116 Strain in Slab Longitudinal Steel, at 6.0 ft. from Support Beam with 50% Debonding (Specimen Set 4)

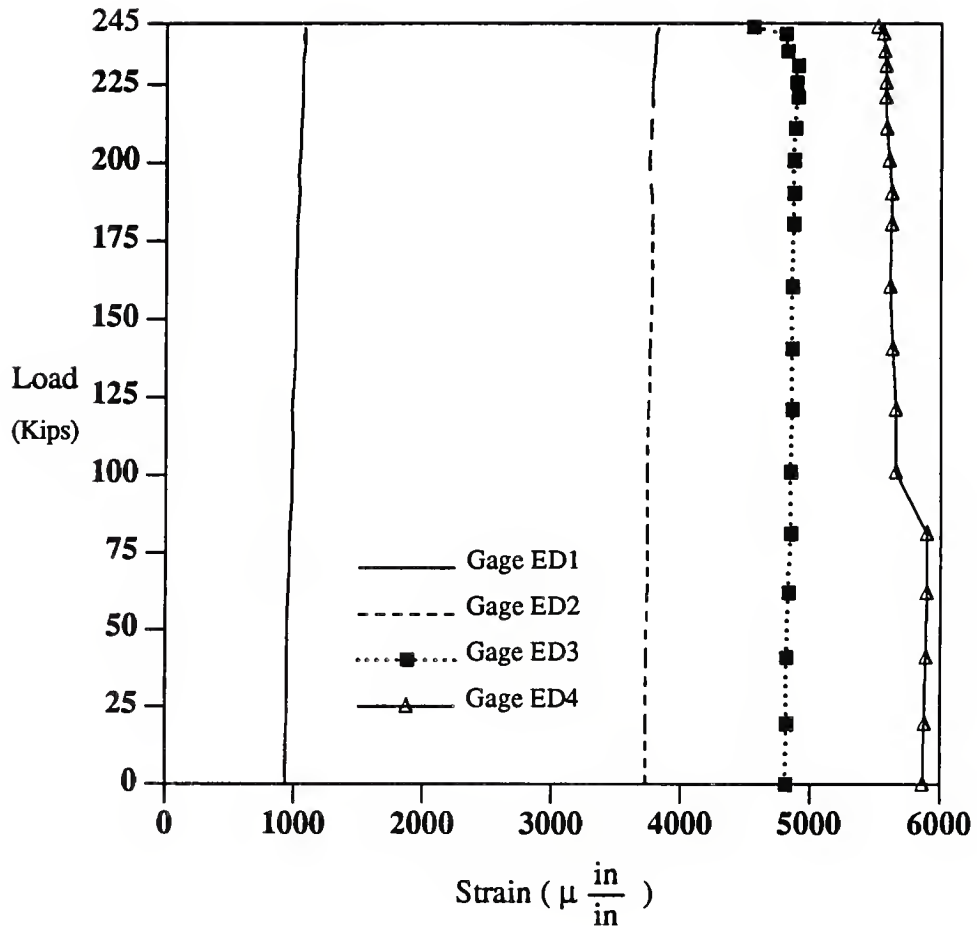


Figure 3.117 Strand Strains at 16 in. from Support
Beam with 0% Debonding (Specimen Set 4)

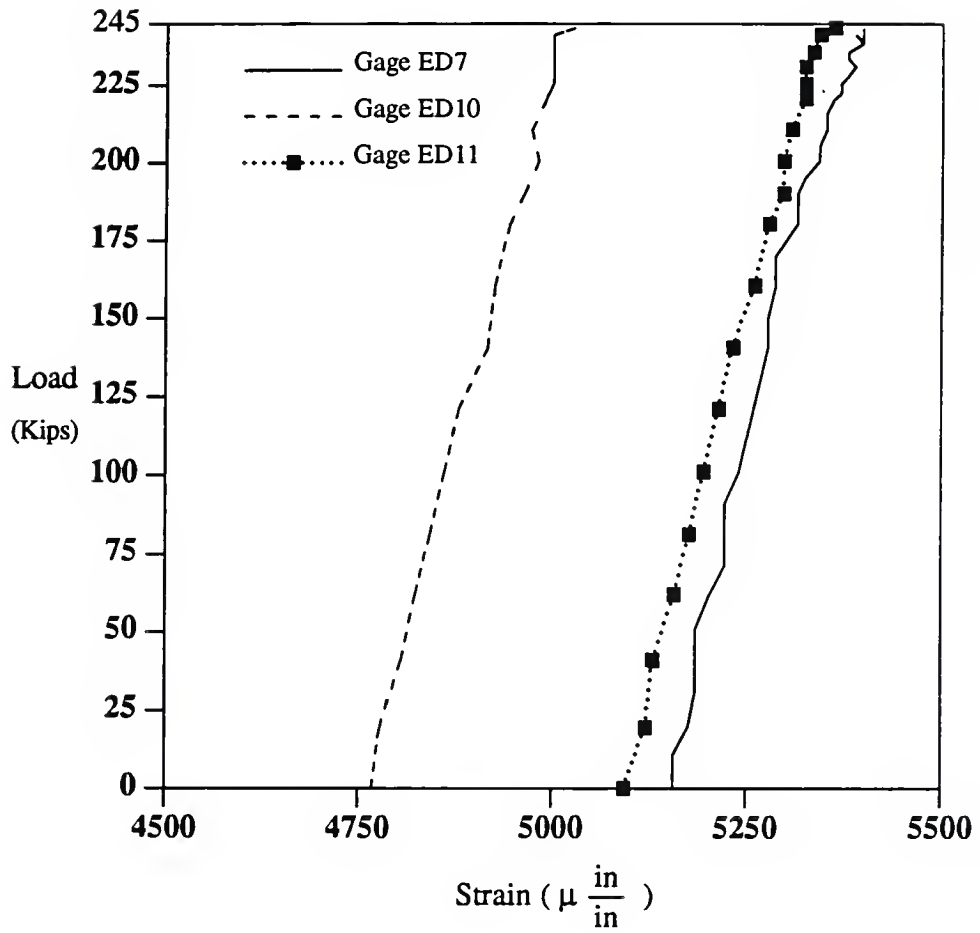


Figure 3.118 Strand Strains at 3.5 ft. from Support
Beam with 0% Debonding (Specimen Set 4)

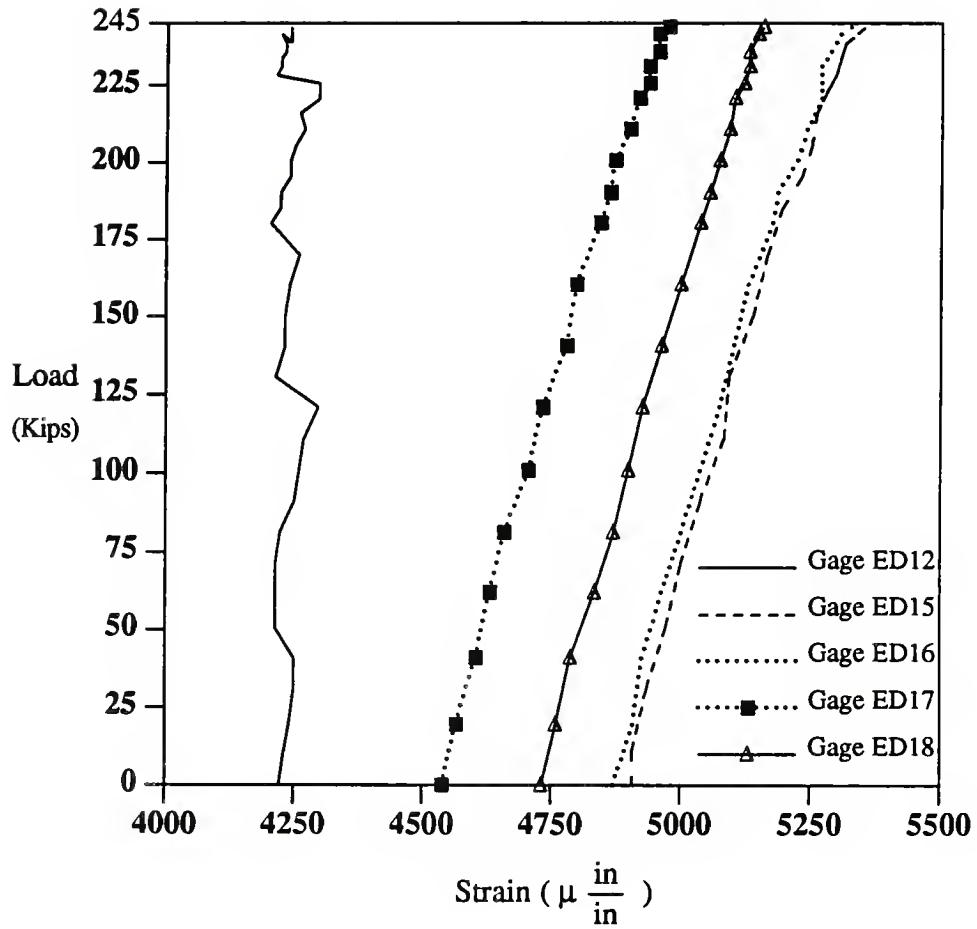


Figure 3.119 Strand Strains at 6.0 ft. from Support
Beam with 0% Debonding (Specimen Set 4)

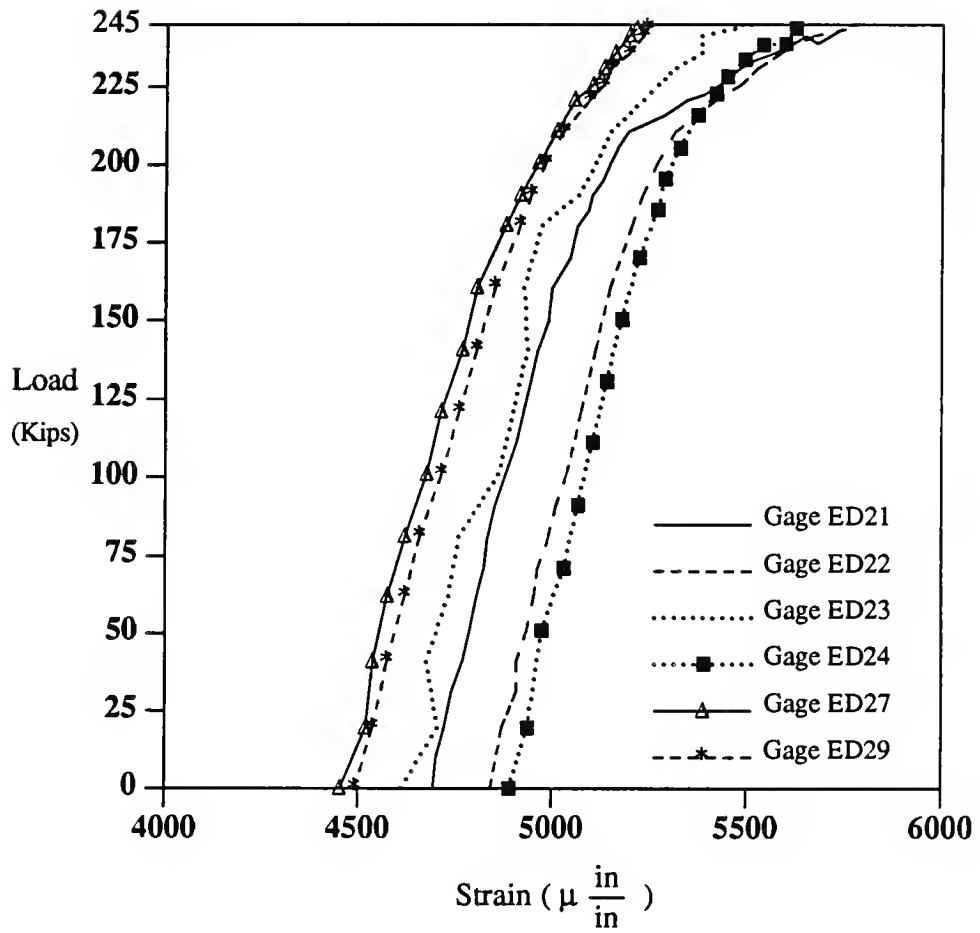


Figure 3.120 Strand Strains at the Point Load Beam with 0% Debonding (Specimen Set 4)

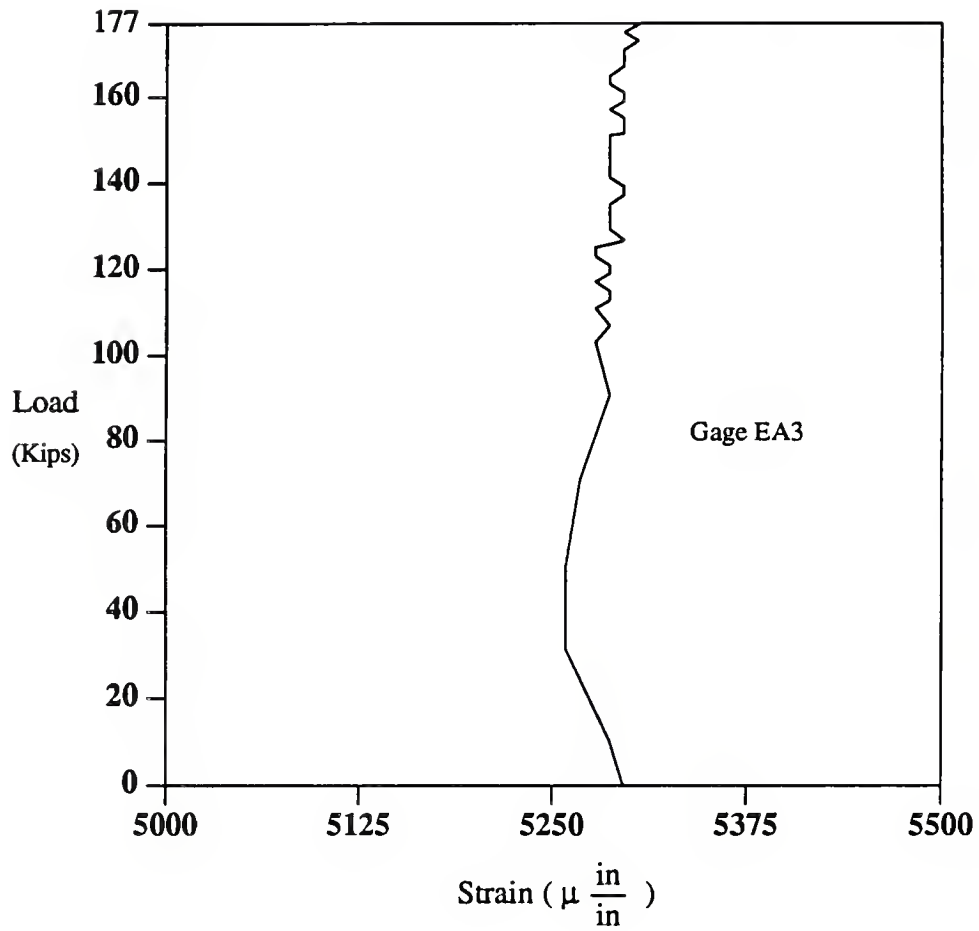


Figure 3.121 Strand Strains at 16 in. from Support
Beam with 50% Debonding (Specimen Set 4)

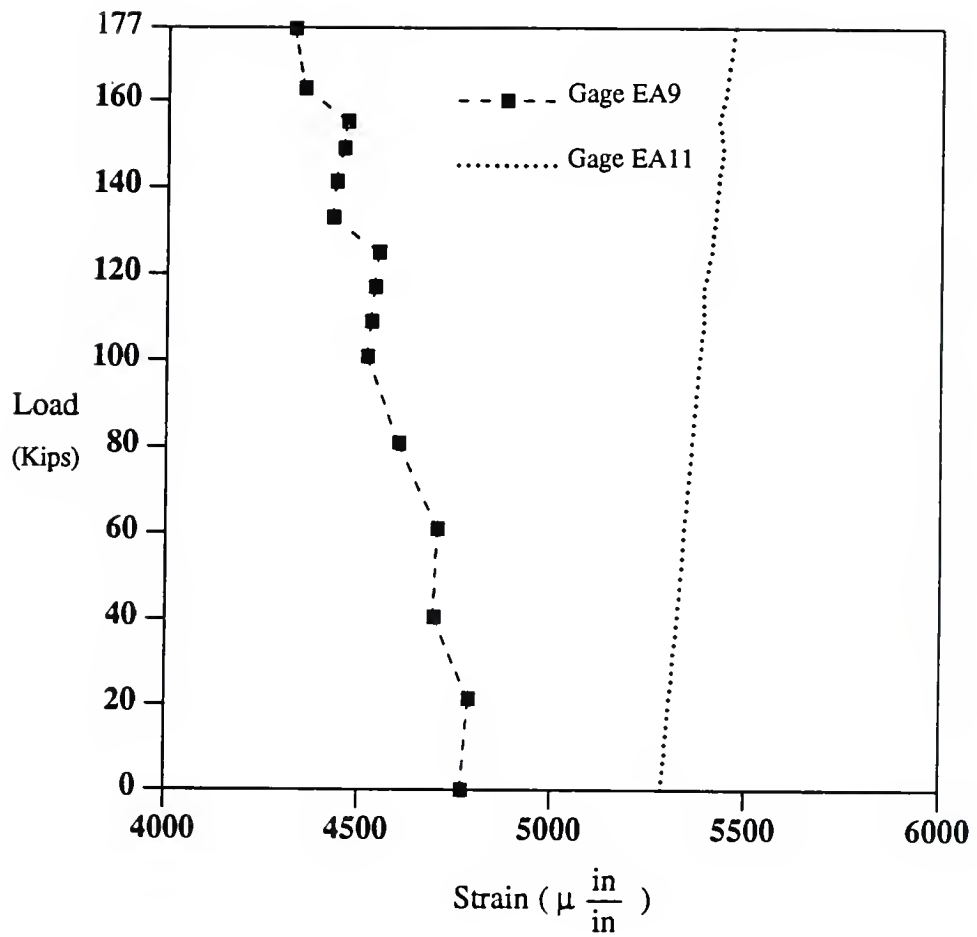


Figure 3.122 Strand Strains at First Debonding Point
Beam with 50% Debonding (Specimen Set 4)

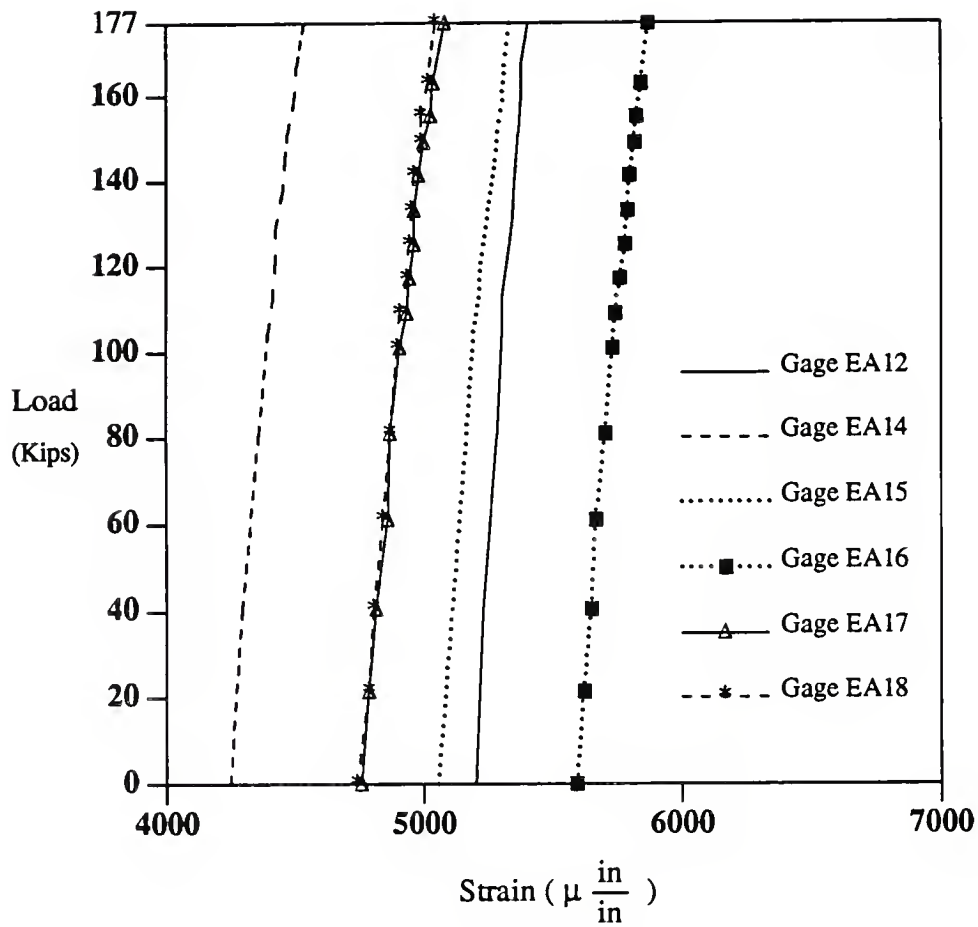


Figure 3.123 Strand Strains at Second Debonding Point
Beam with 50% Debonding (Specimen Set 4)

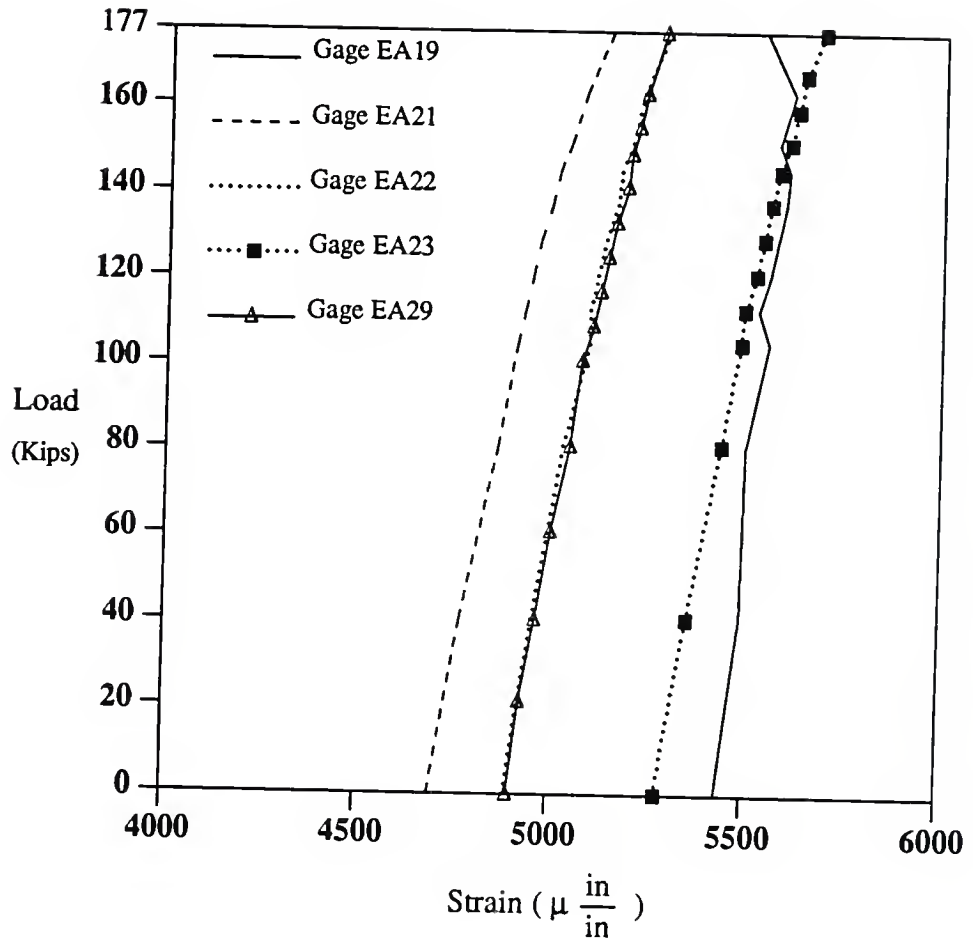


Figure 3.124 Strand Strains at the Point Load Beam with 50% Debonding (Specimen Set 4)

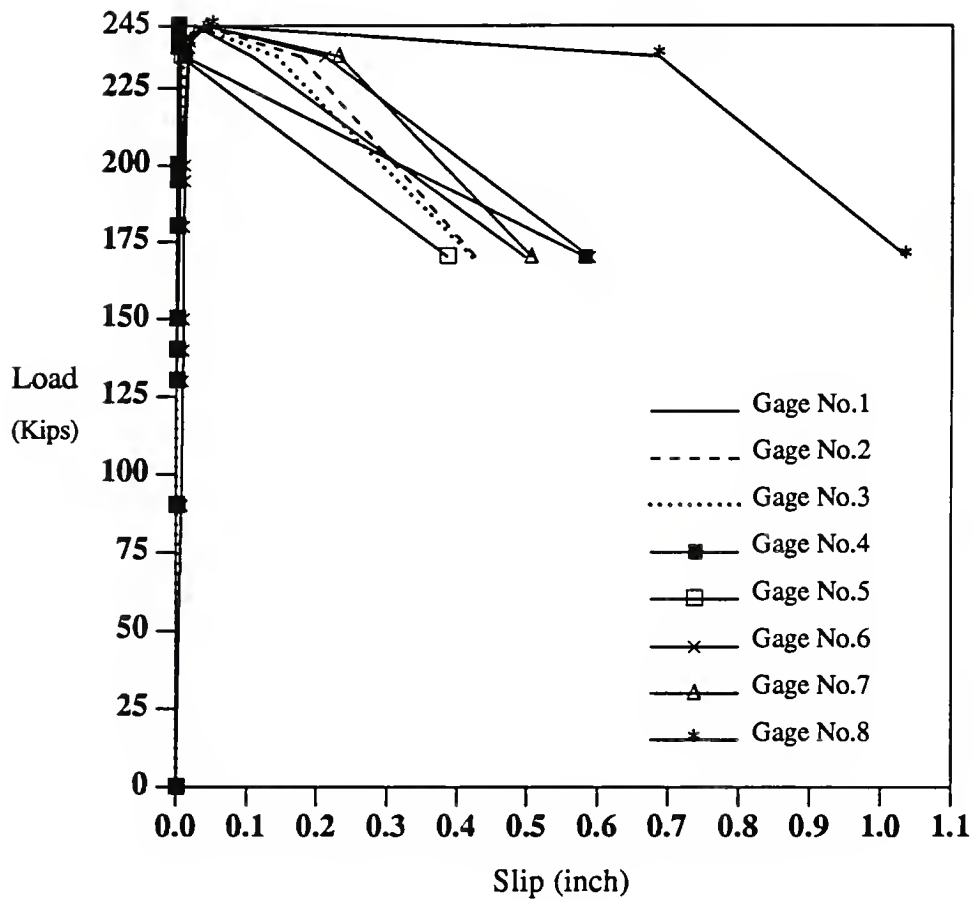


Figure 3.125 Strand Slip, Beam with 0% Debonding
Specimen Set 4

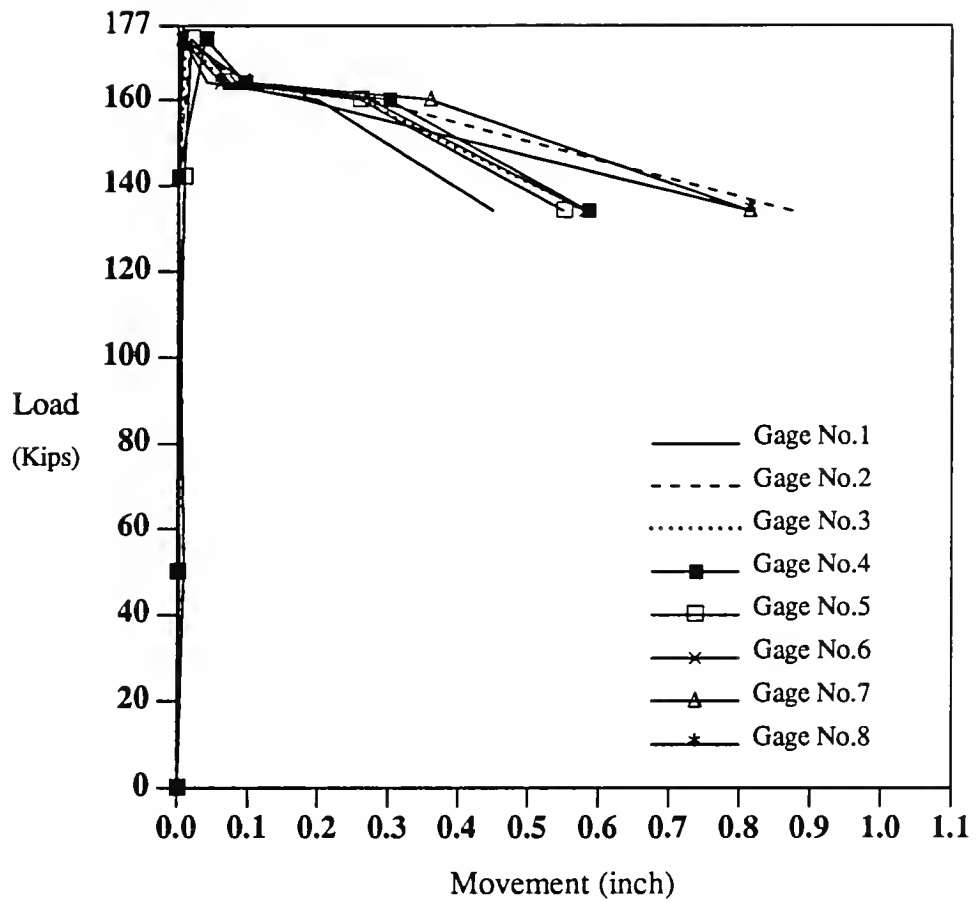
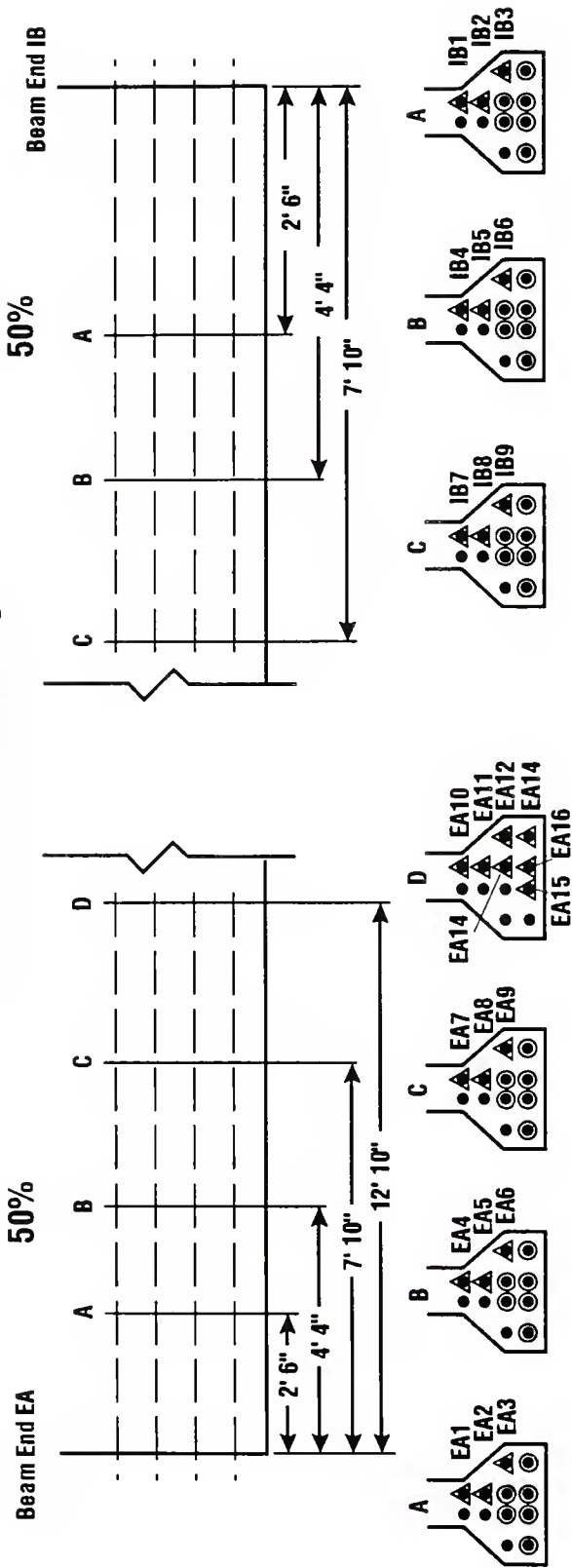


Figure 3.126 Strand Movement, Beam with 50% Debonding
Specimen Set 4

Beam With Debonding



Beam With 0% Debonding

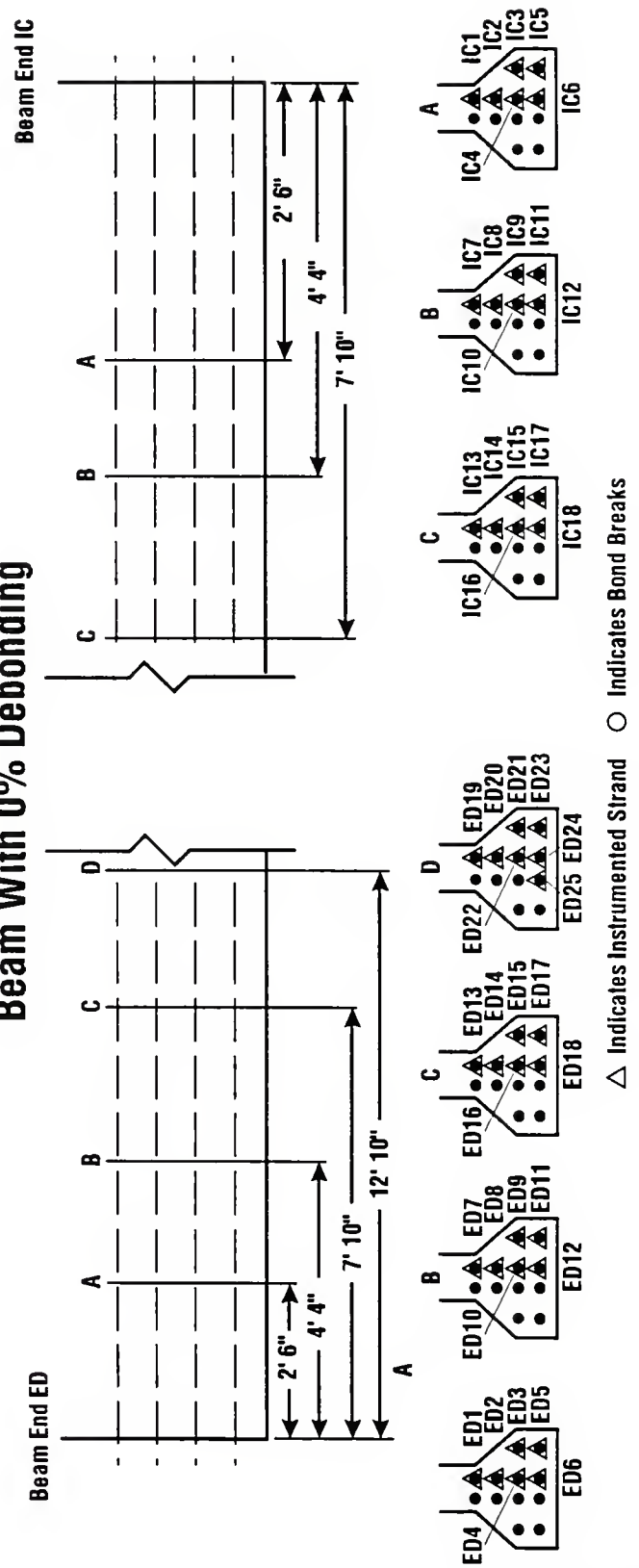


Figure (3.127) Strand Debonding Scheme and Instrumentation for Specimen Set 5.

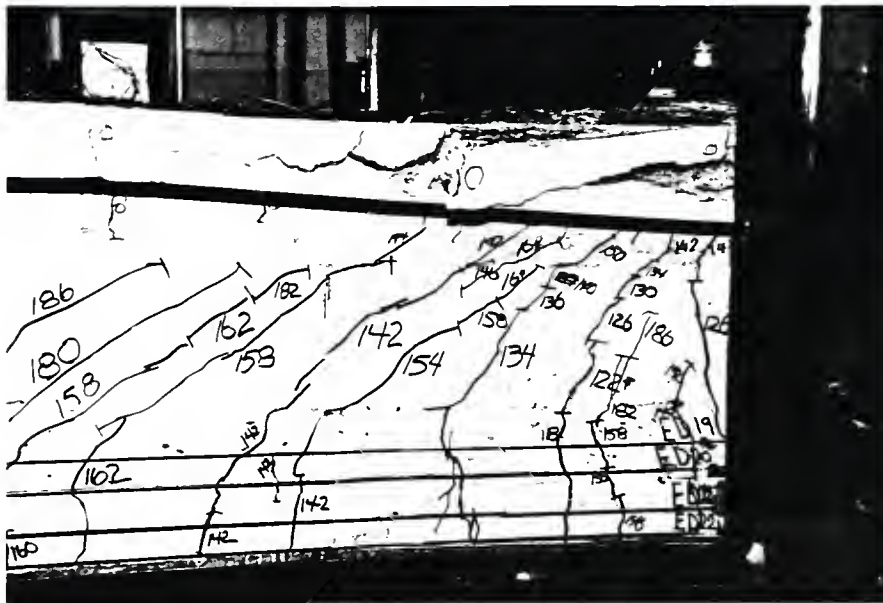


**Figure 3.128 Crack Pattern of 0% Debonded Beam at Failure
Specimen Set 5**

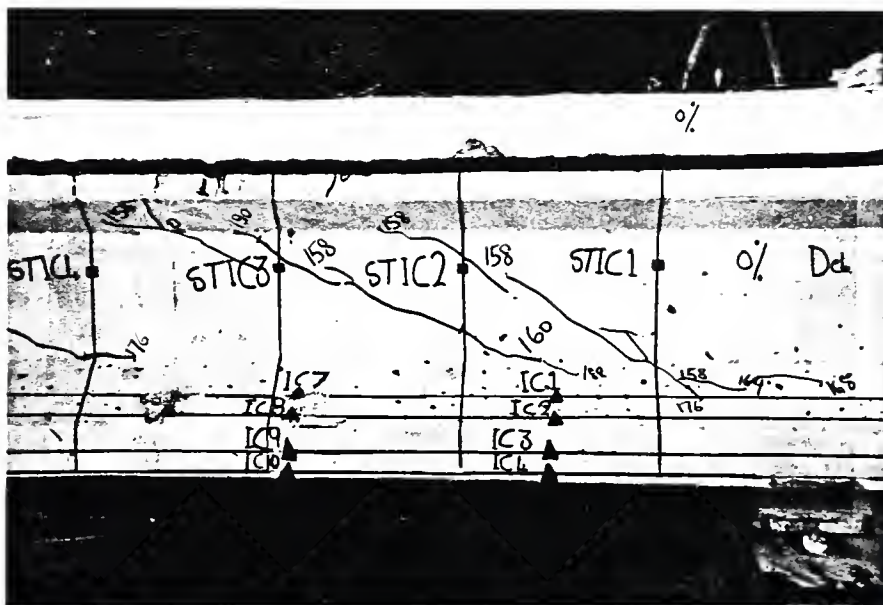


**Figure 3.129 Web-Shear Cracking at Right Support of 0% Debonded Beam
Specimen Set 5**

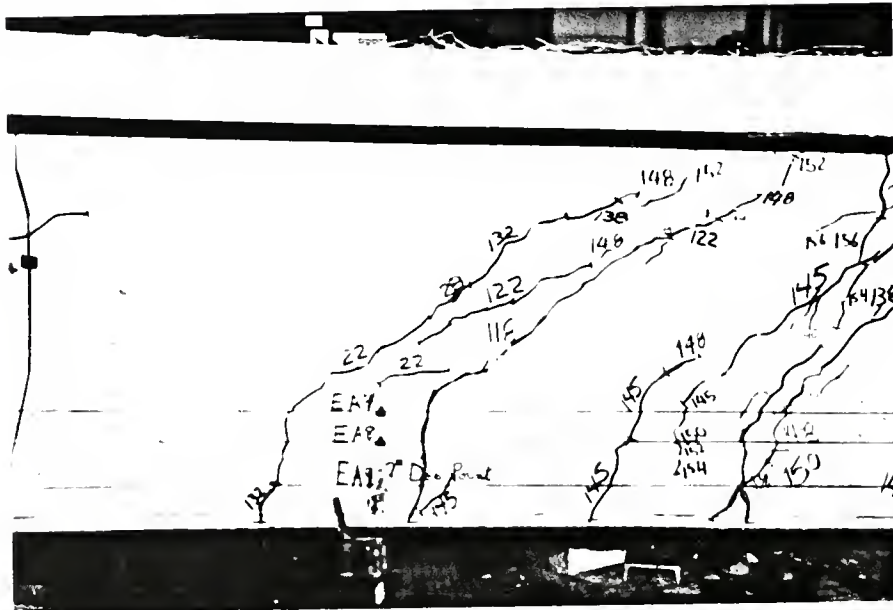


Figure 3.105 Flexure-Shear Cracking at Debonding Point
Specimen Set 5

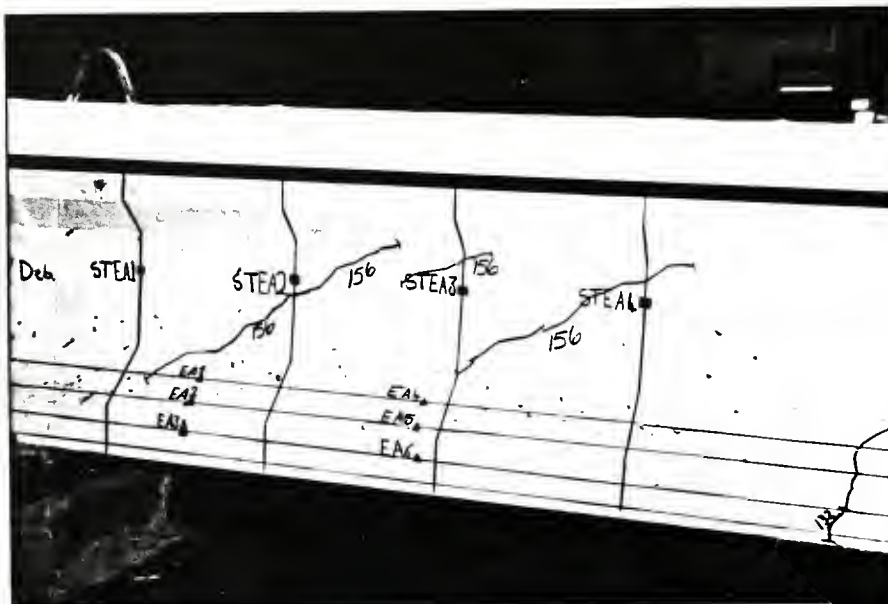


Figure 3.131 Web-Shear Cracking at Left Support of 50% Debonded Beam
Specimen Set 5

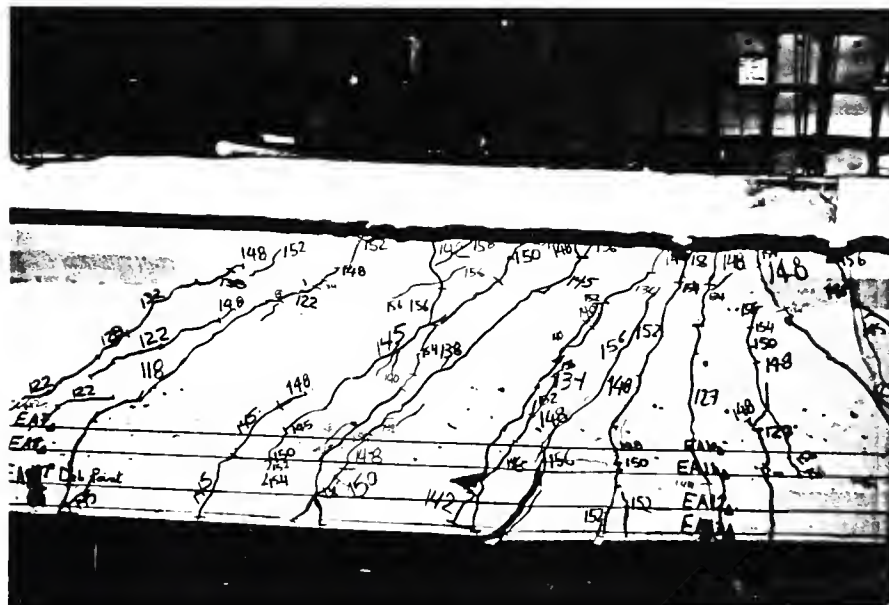


Figure 3.132 Crack Pattern of 50% Debonded Beam at Failure
Specimen Set 5

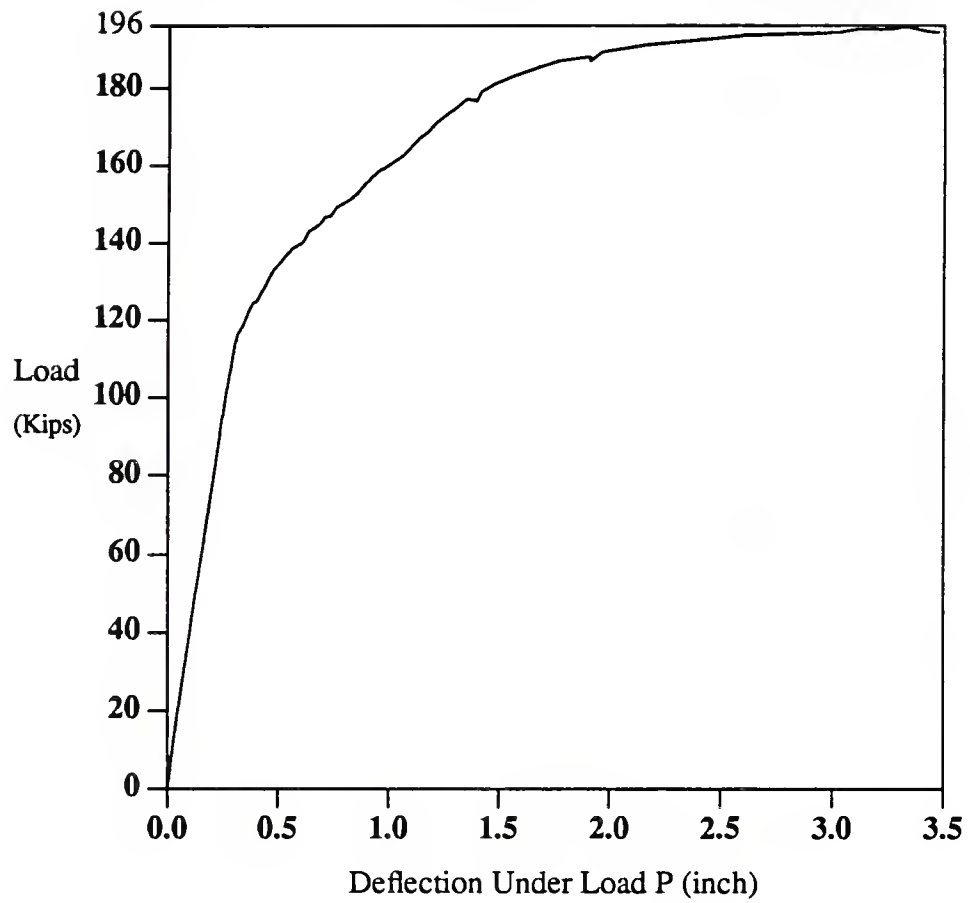


Figure 3.133 Load-Deflection Relationship, Beam with 0% Debonding
(Specimen Set 5)

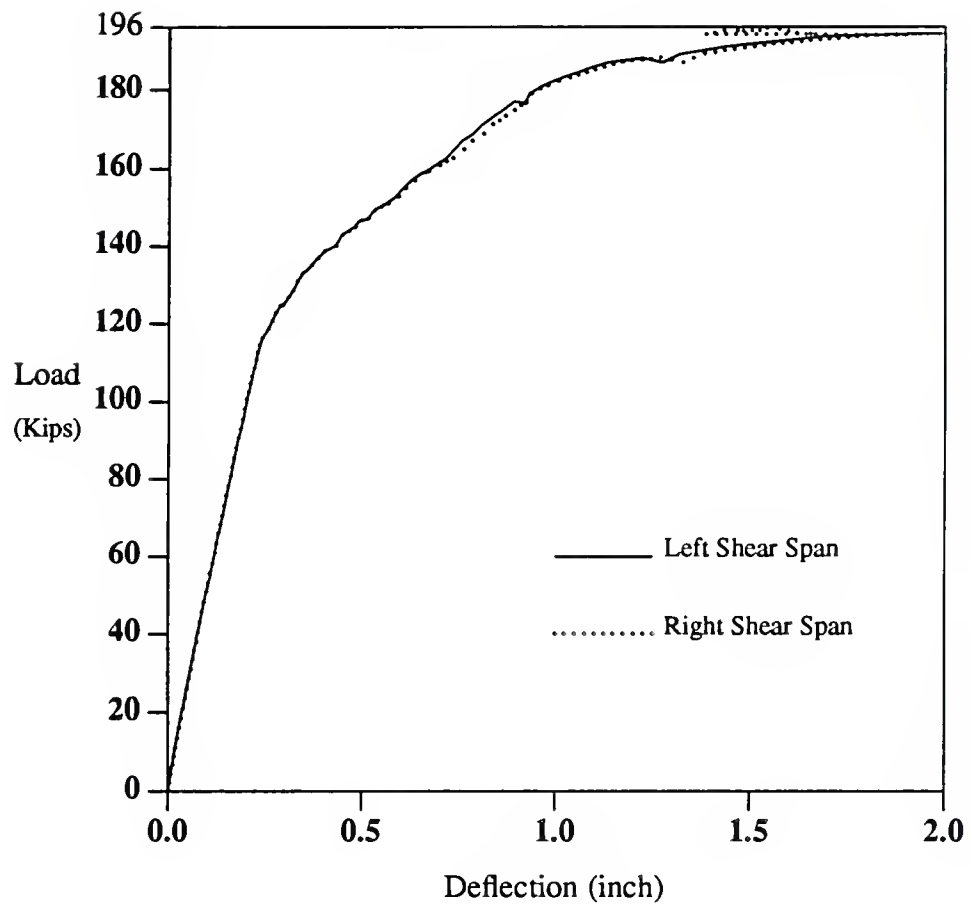


Figure 3.134 Load-Deflection Relationship at 7 ft. from Support
Beam with 0% Debonding (Specimen Set 5)

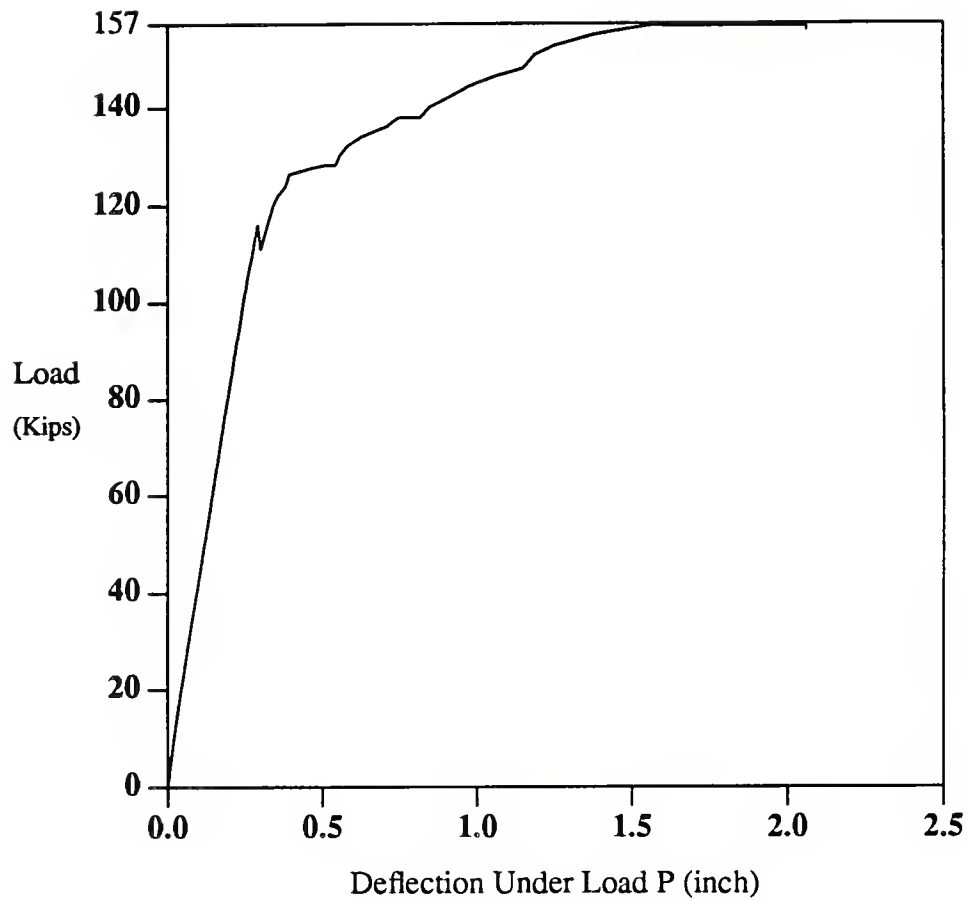


Figure 3.135 Load-Deflection Relationship, Beam with 50% Debonding
(Specimen Set 5)

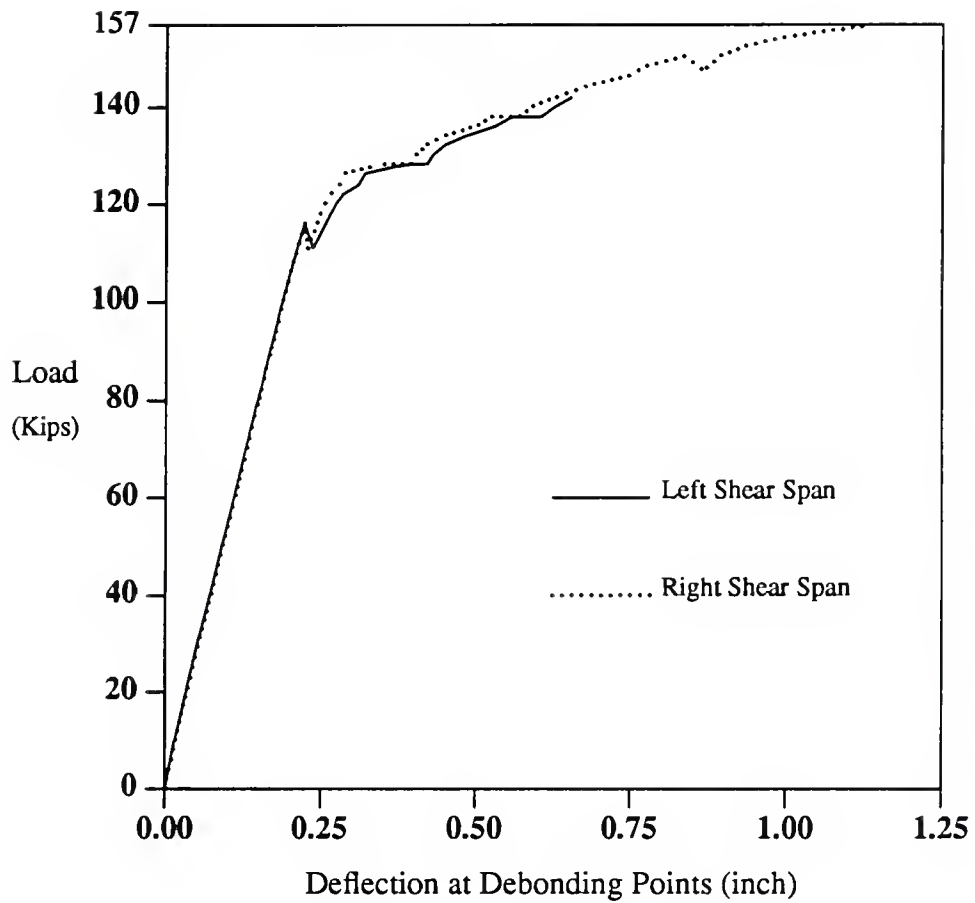


Figure 3.136 Load-Deflection Relationship, Beam with 50% Debonding
(Specimen Set 5)

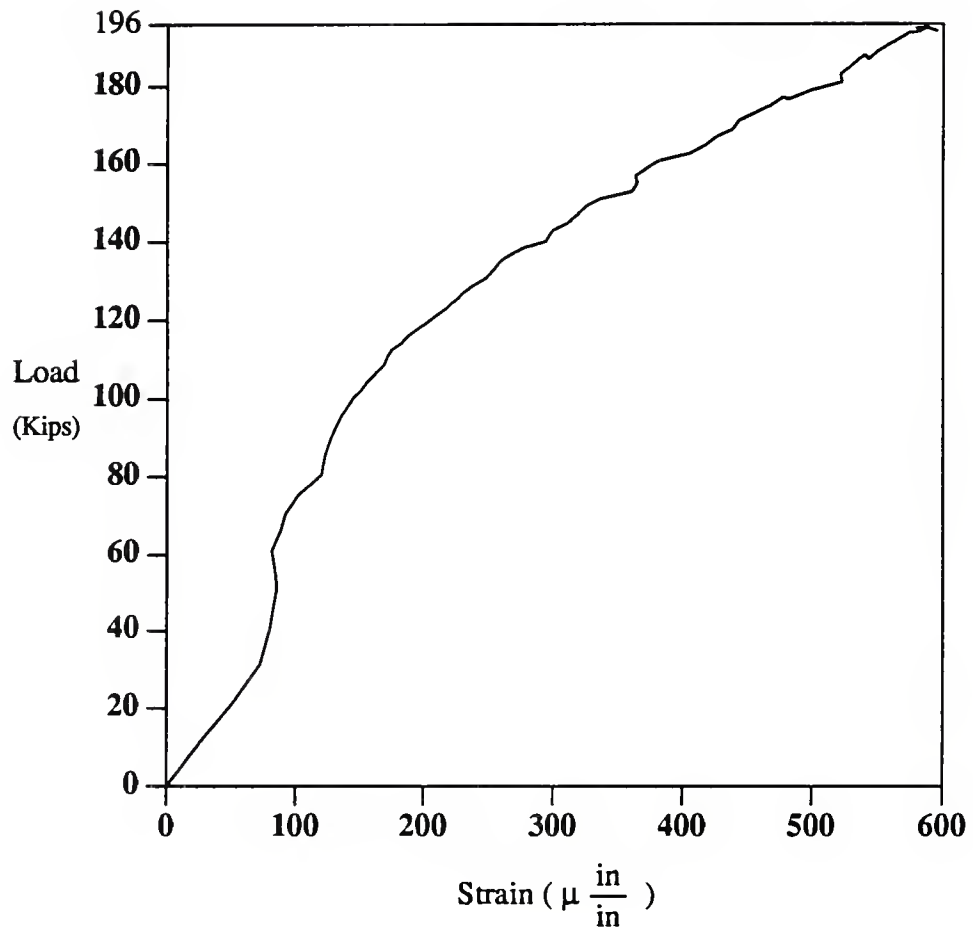


Figure 3.137 Surface Strain in Slab at the 7 ft. from Support
Beam with 0% Debonding (Specimen Set 5)

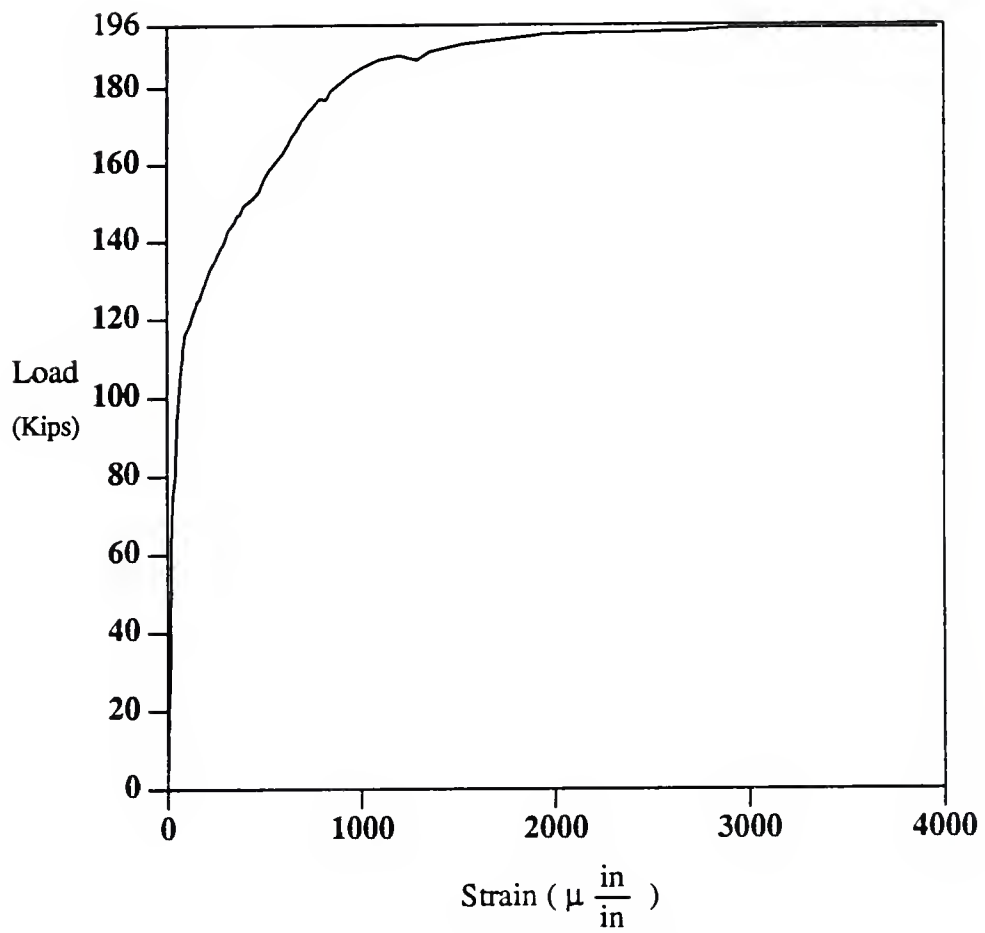


Figure 3.138 Surface Strain in Slab. at the Point Load
Beam with 0% Debonding (Specimen Set 5)

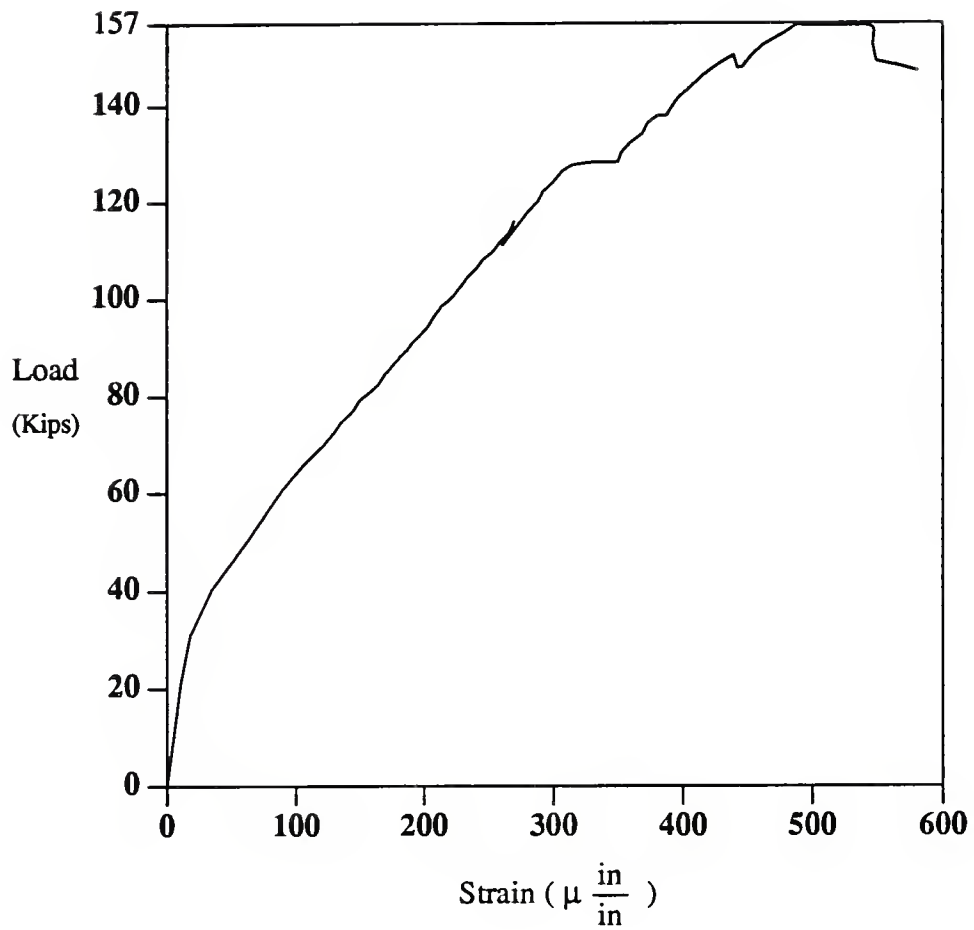


Figure 3.139 Surface Strain in Slab, at Debonding Point
Beam with 50% Debonding (Specimen Set 5)

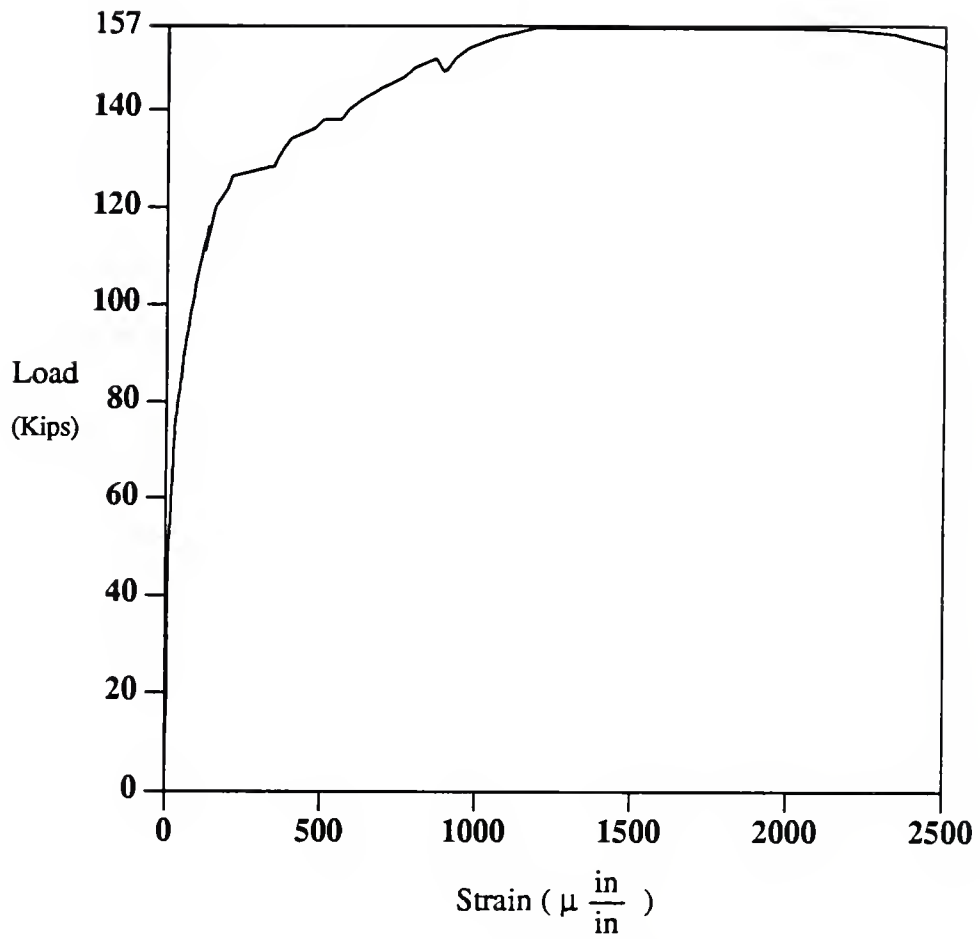


Figure 3.140 Surface Strain in Slab at the Point Load
Beam with 50% Debonding (Specimen Set 5)

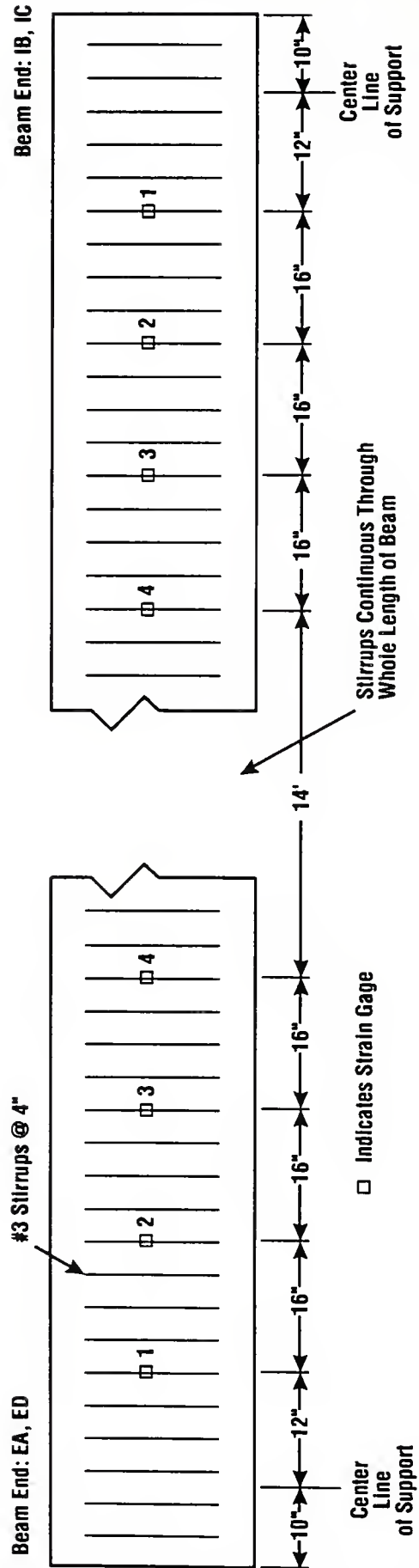


Figure (3.141) Location of Stirrup Reinforcement and Instrumentation for Specimen Set 5.

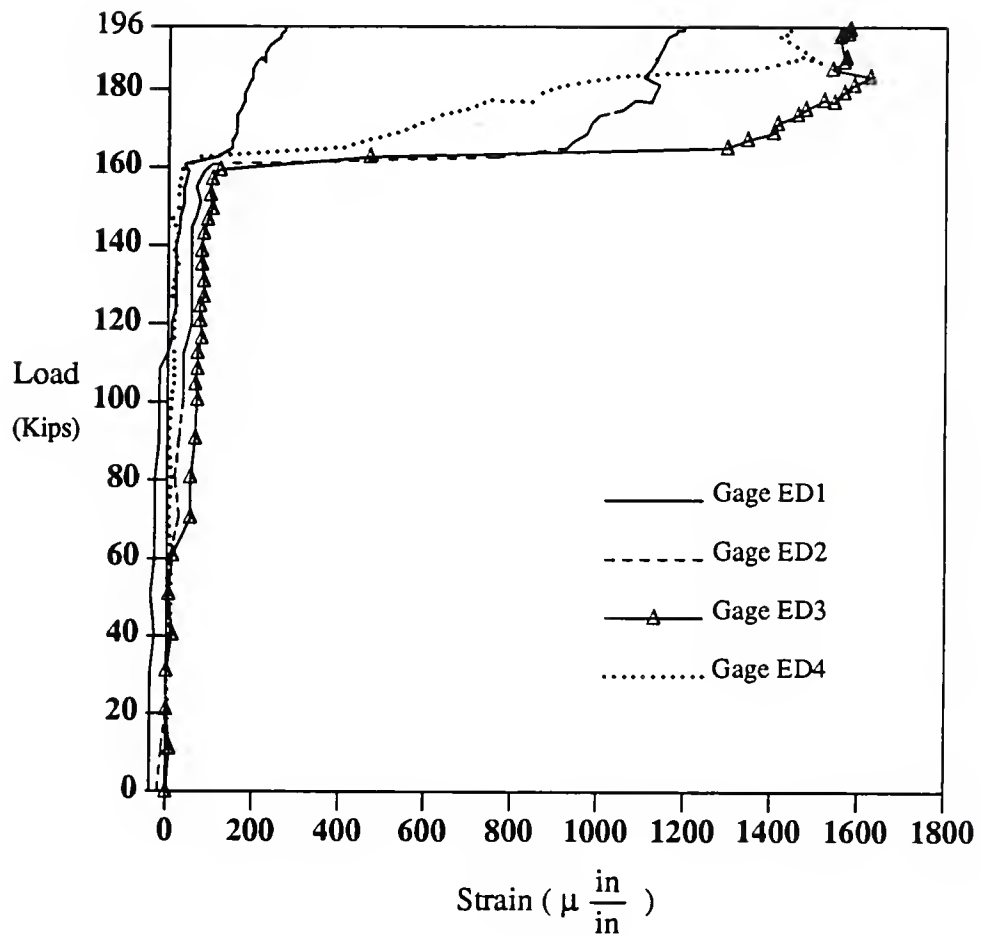


Figure 3.142 Stirrup Strain in Left Shear Span
Beam with 0% Debonding (Specimen Set 5)

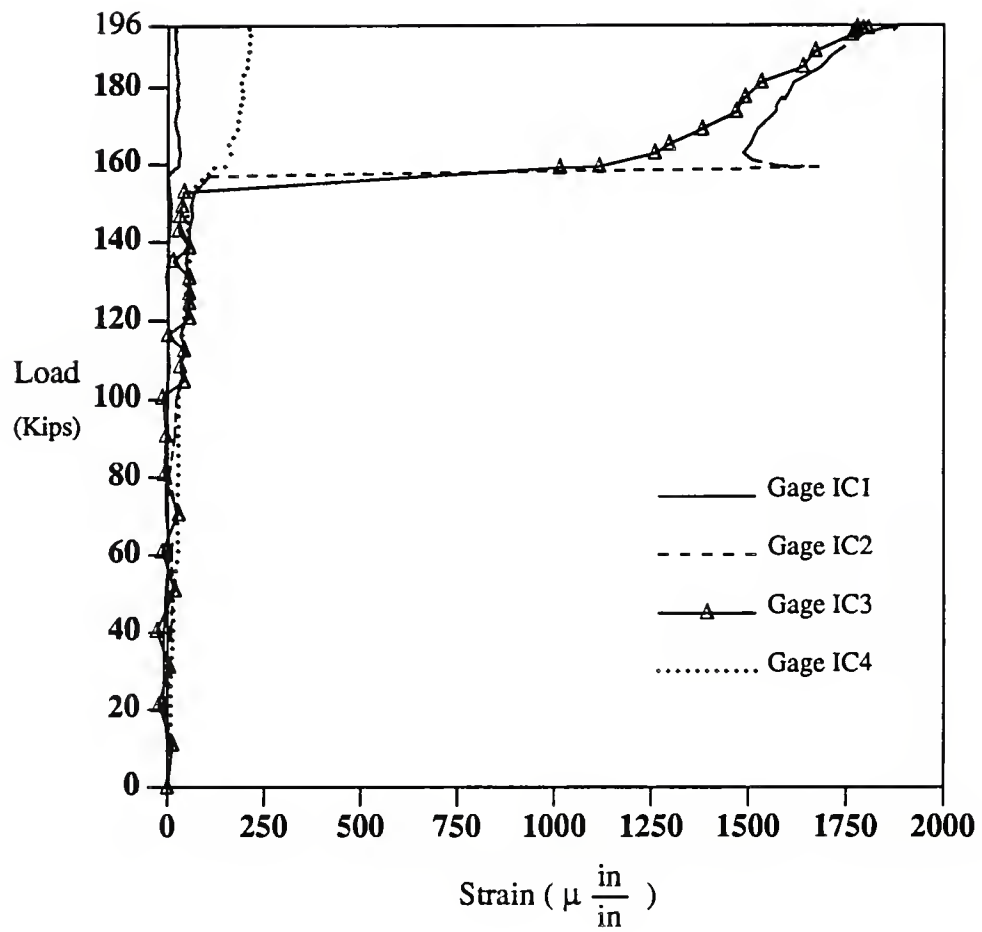


Figure 3.143 Stirrup Strain in Right Shear Span Beam with 0% Debonding (Specimen Set 5)

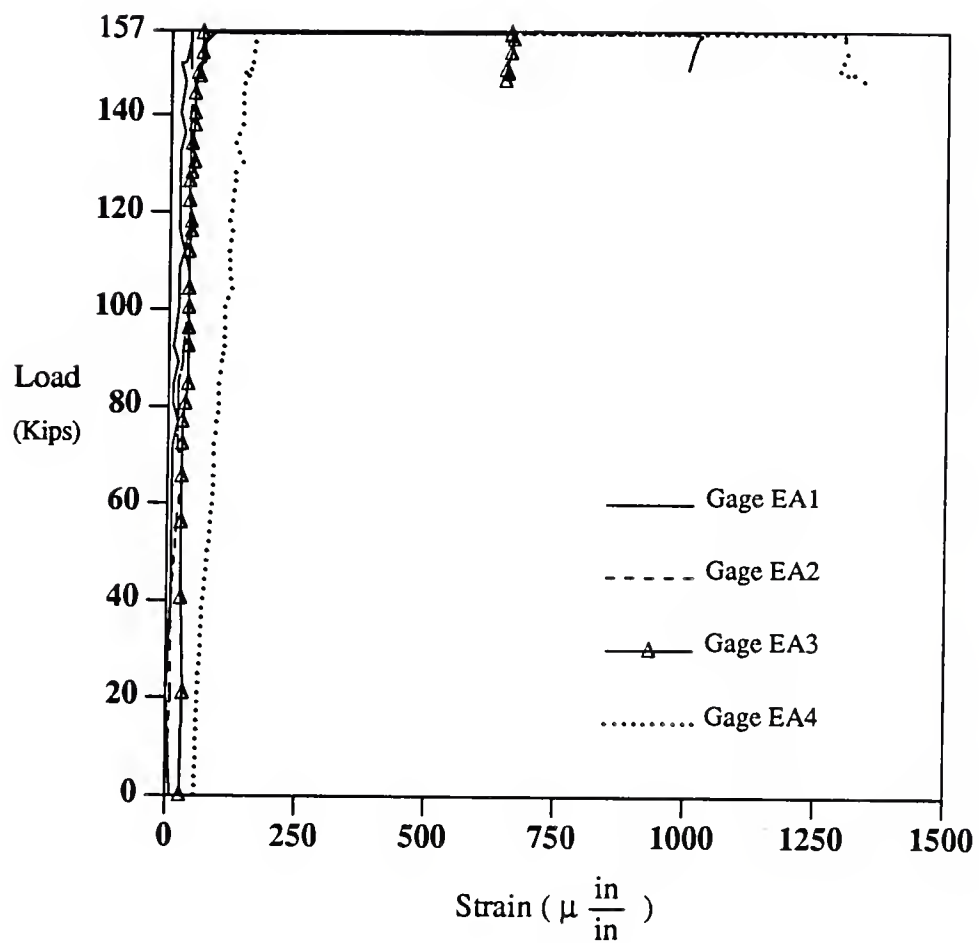


Figure 3.144 Stirrup Strain in Left Shear Span Beam with 50% Debonding (Specimen Set 5)

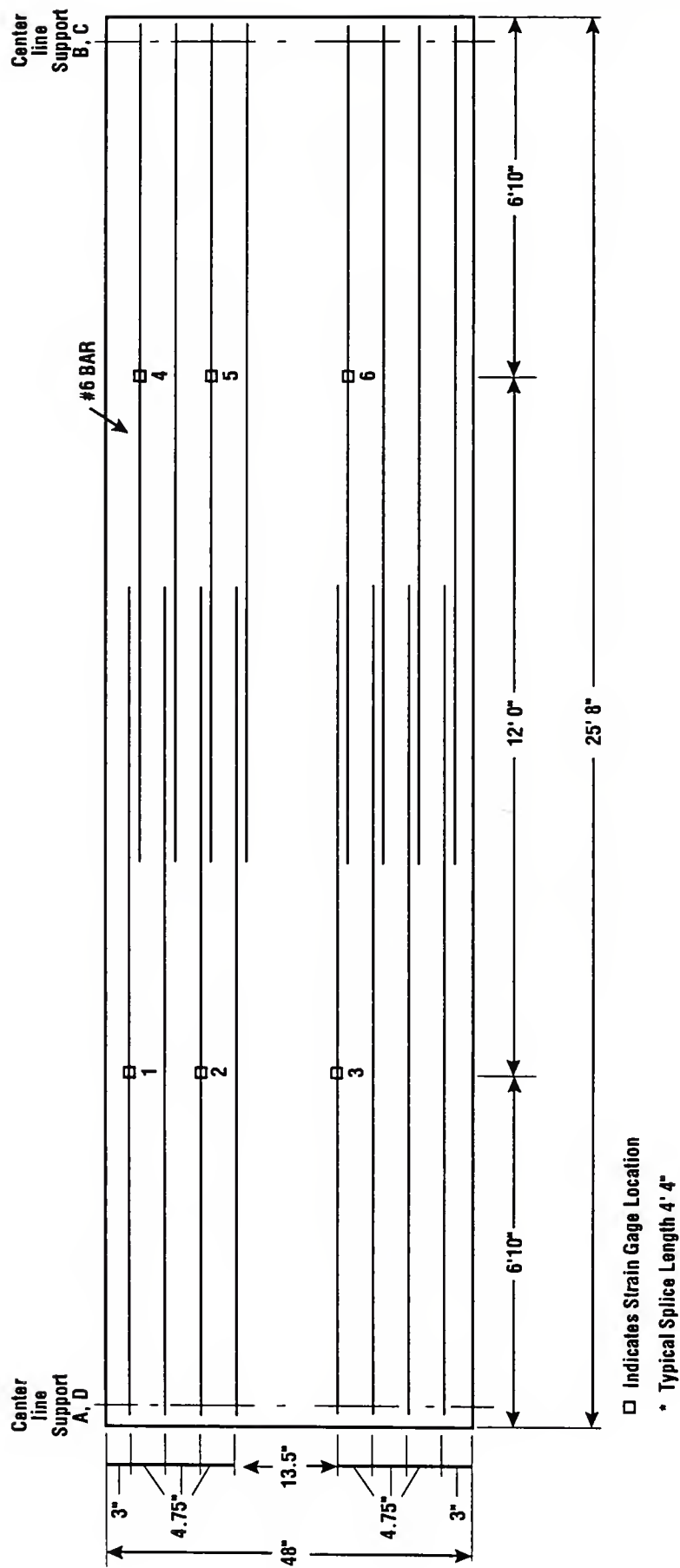


Figure (3.145) Slab Longitudinal Steel Instrumentation for Specimen Set 5.

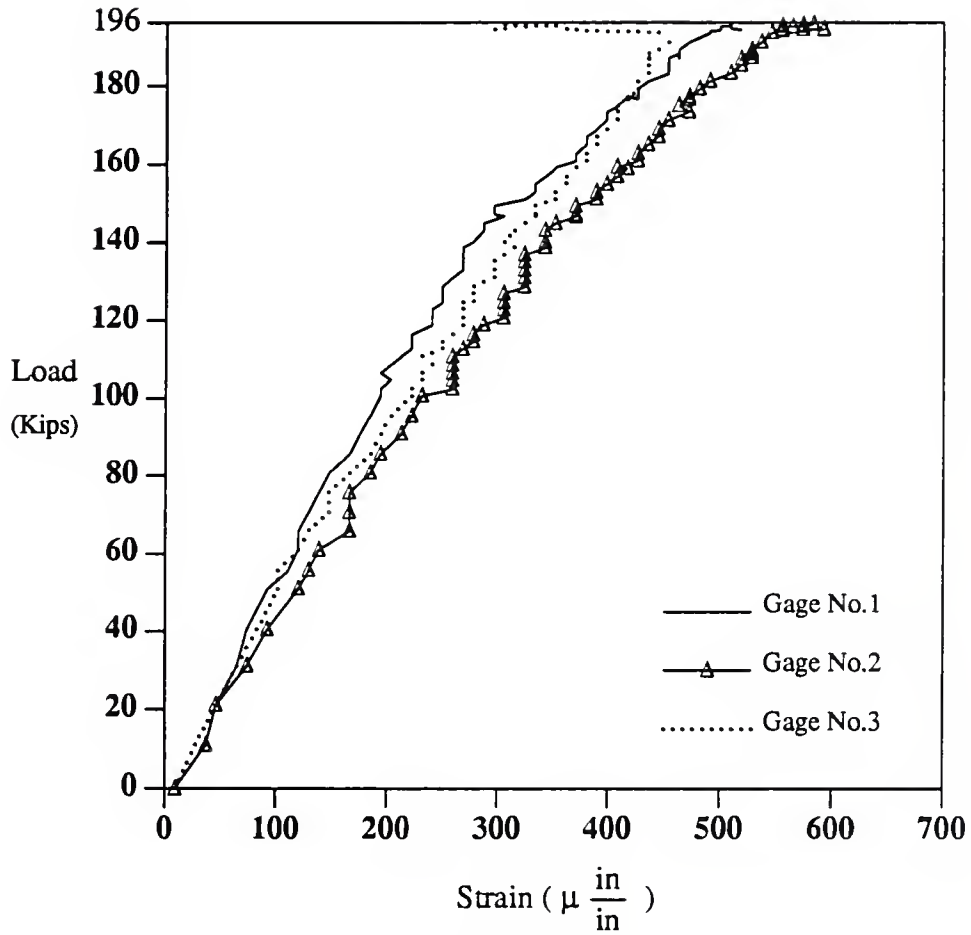


Figure 3.146 Slab Steel Strain in Left Shear Span
Beam with 0% Debonding (Specimen Set 5)

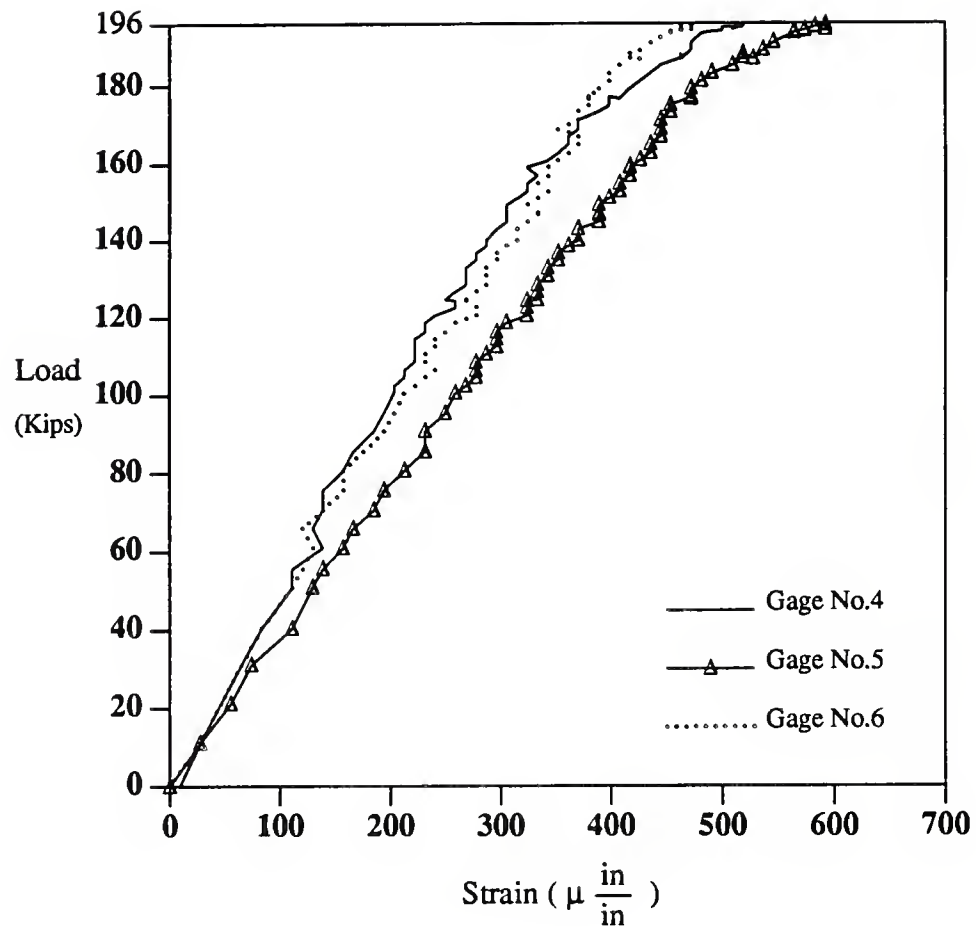


Figure 3.147 Slab Steel Strain in Right Shear Span
Beam with 0% Debonding (Specimen Set 5)

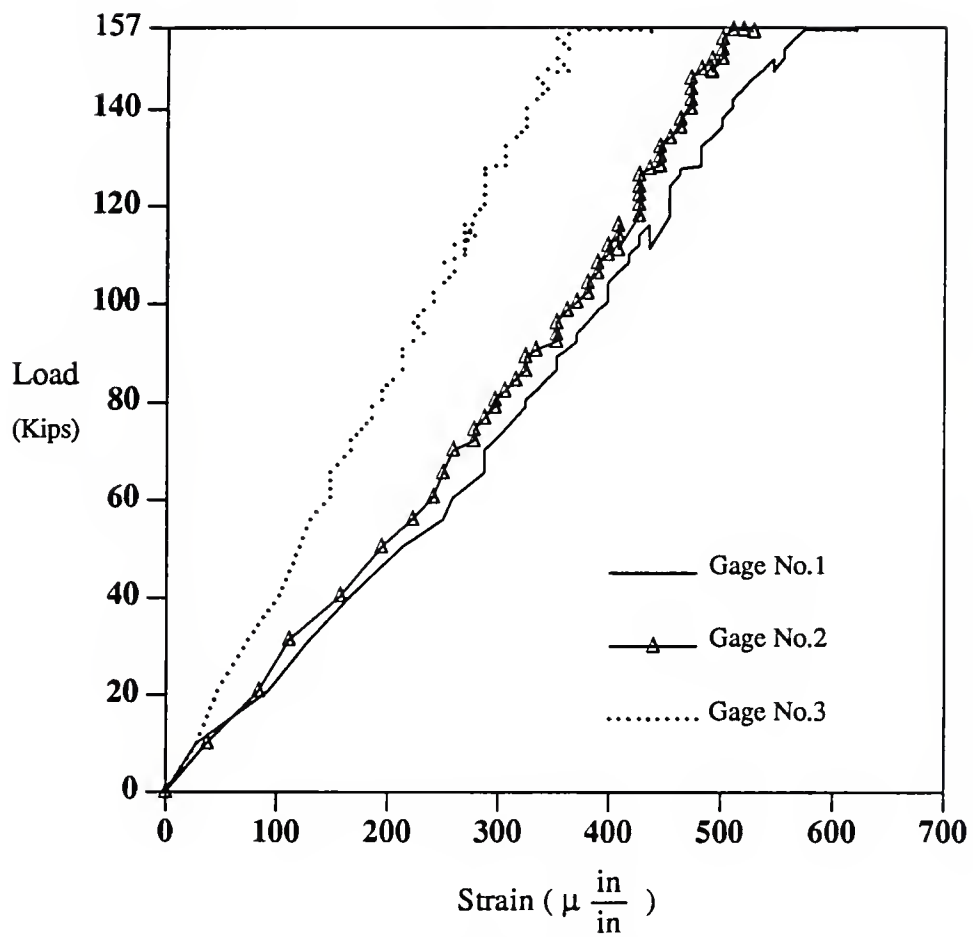


Figure 3.148 Slab Steel Strain in Left Shear Span
Beam with 50% Debonding (Specimen Set 5)

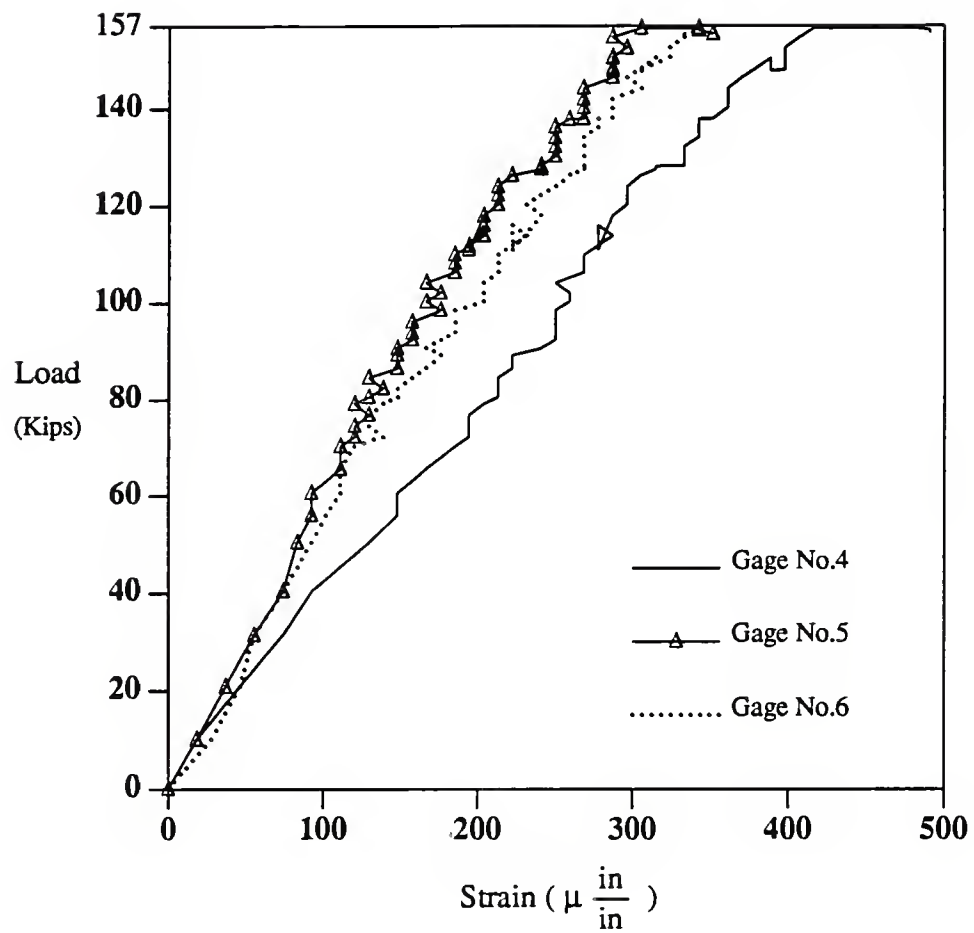


Figure 3.149 Slab Steel Strain in Right Shear Span
Beam with 50% Debonding (Specimen Set 5)

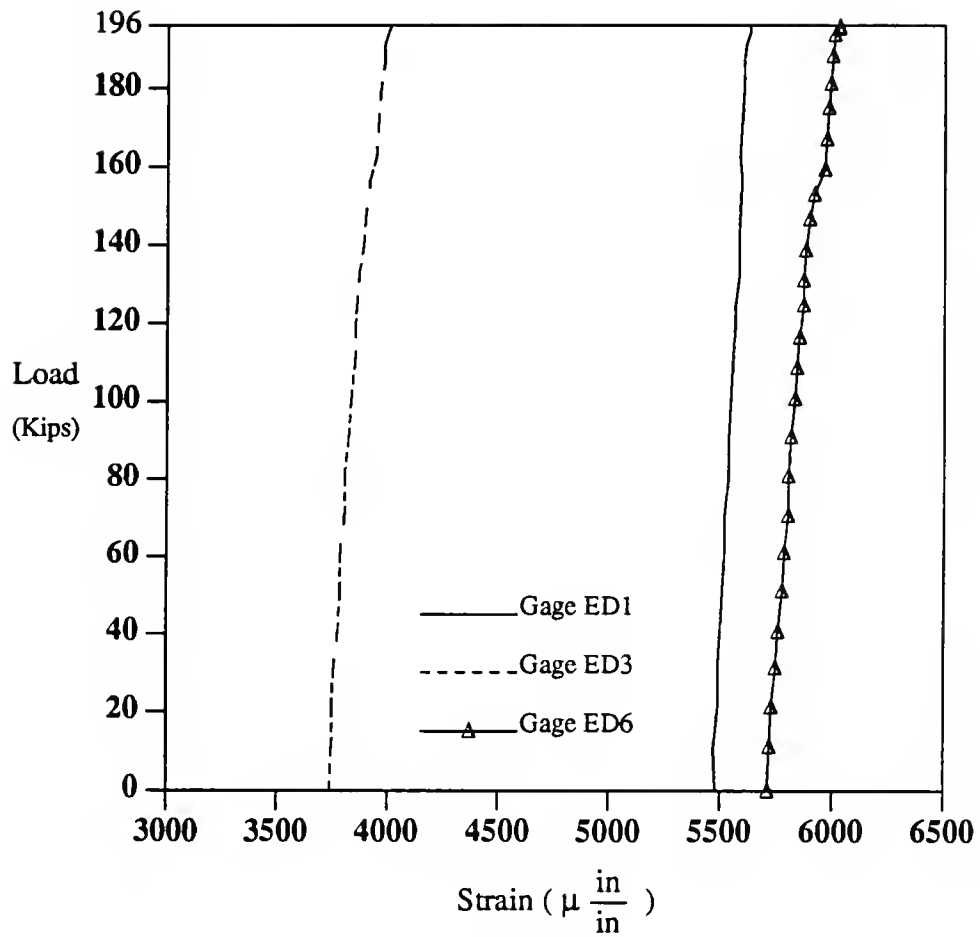


Figure 3.150 Strand Strain at 20 in. from Support ED
Beam with 0% Debonding (Specimen Set 5)

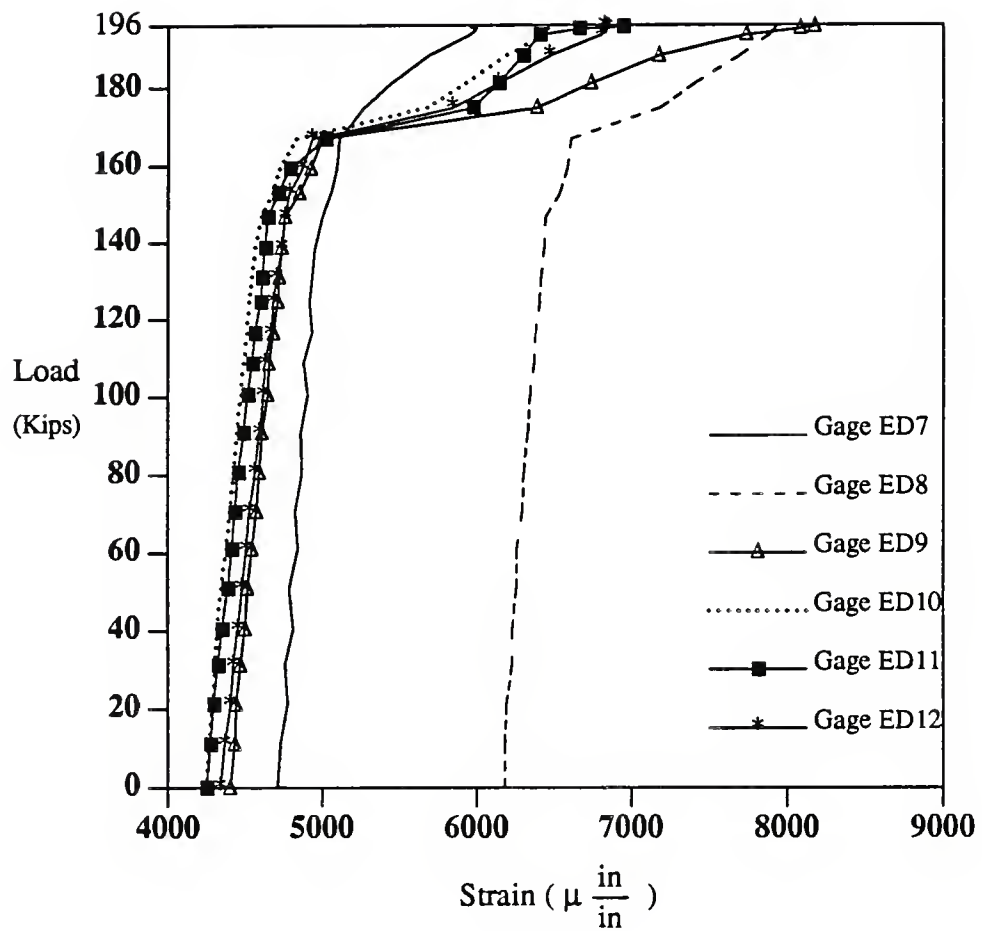


Figure 3.151 Strand Strain at 42 in. from Support ED
Beam with 0% Debonding (Specimen Set 5)

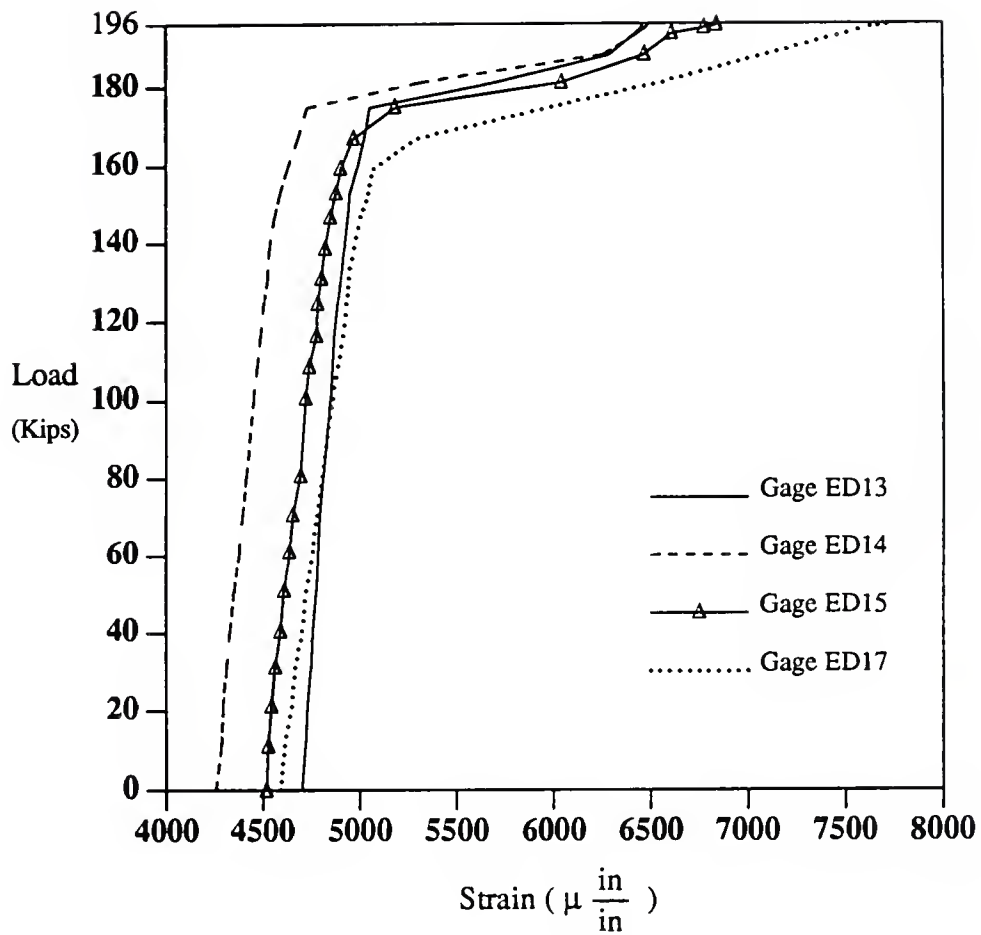


Figure 3.152 Strand Strain at 84 in. from Support ED
Beam with 0% Debonding (Specimen Set 5)

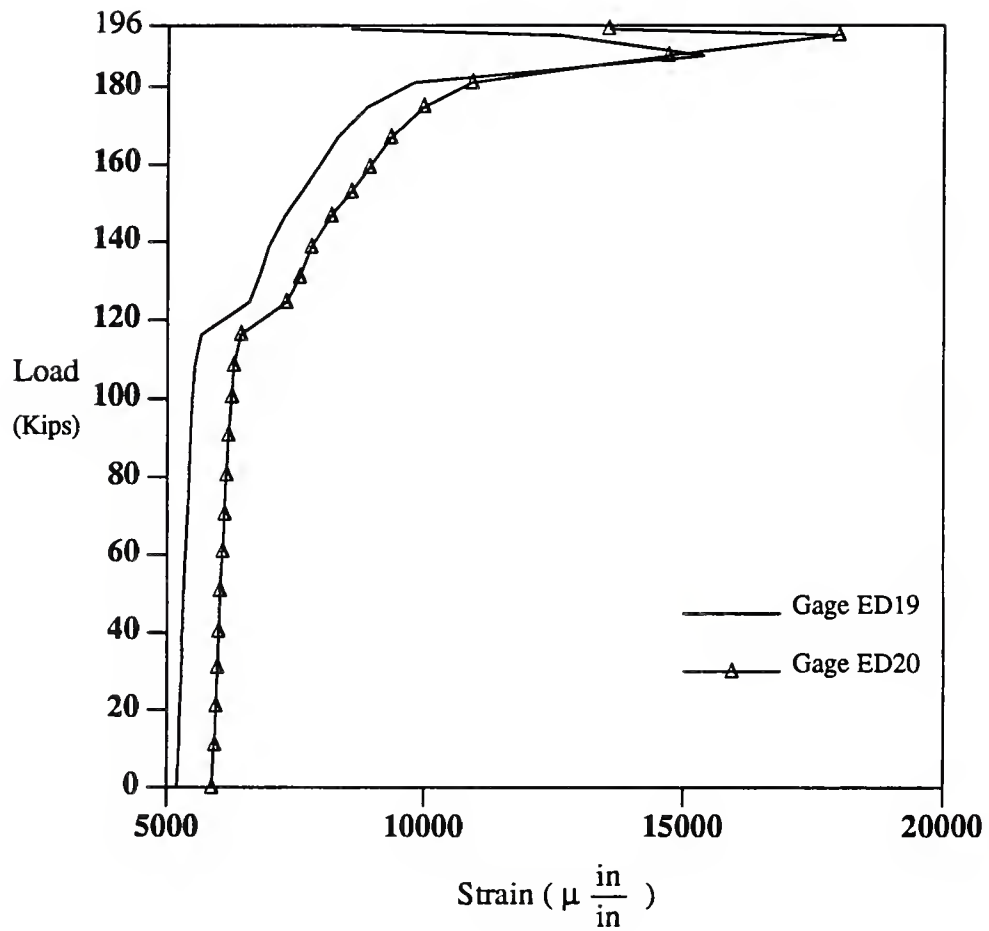


Figure 3.153 Strand Strain at the Point Load Beam with 0% Debonding (Specimen Set 5)

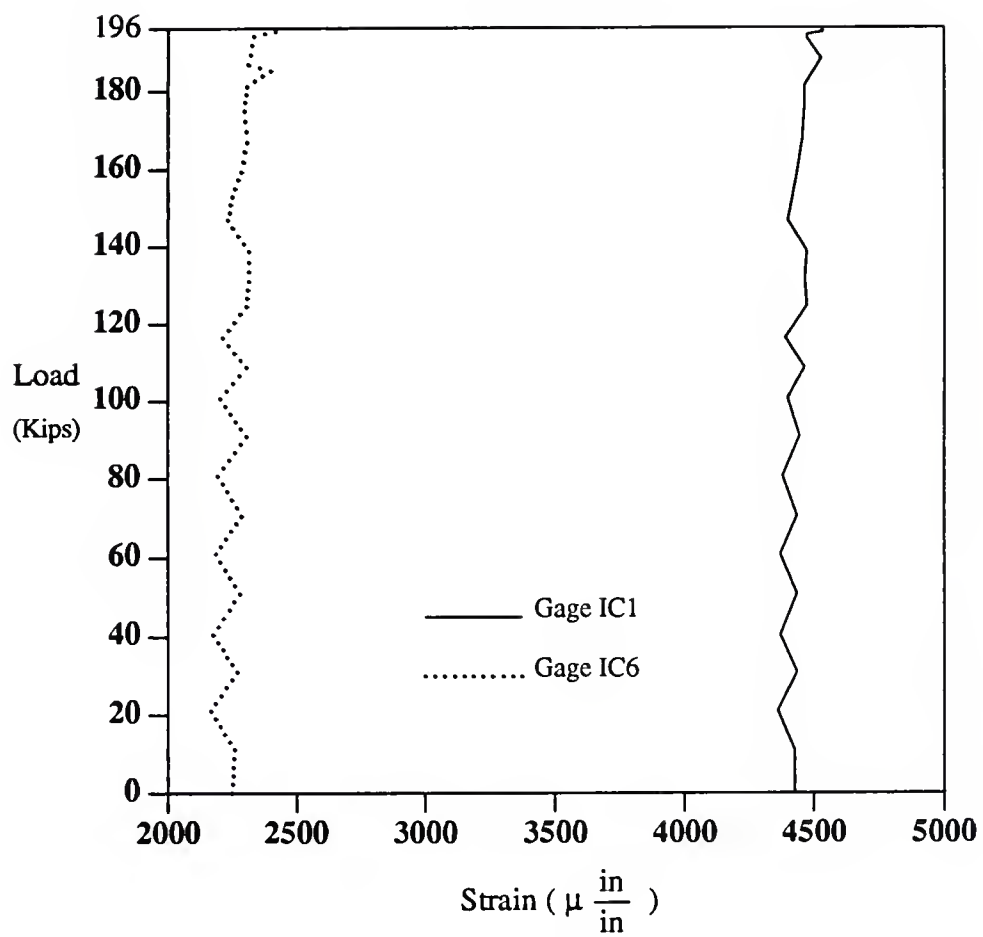


Figure 3.154 Strand Strain at 20 in. from Support IC
Beam with 0% Debonding (Specimen Set 5)

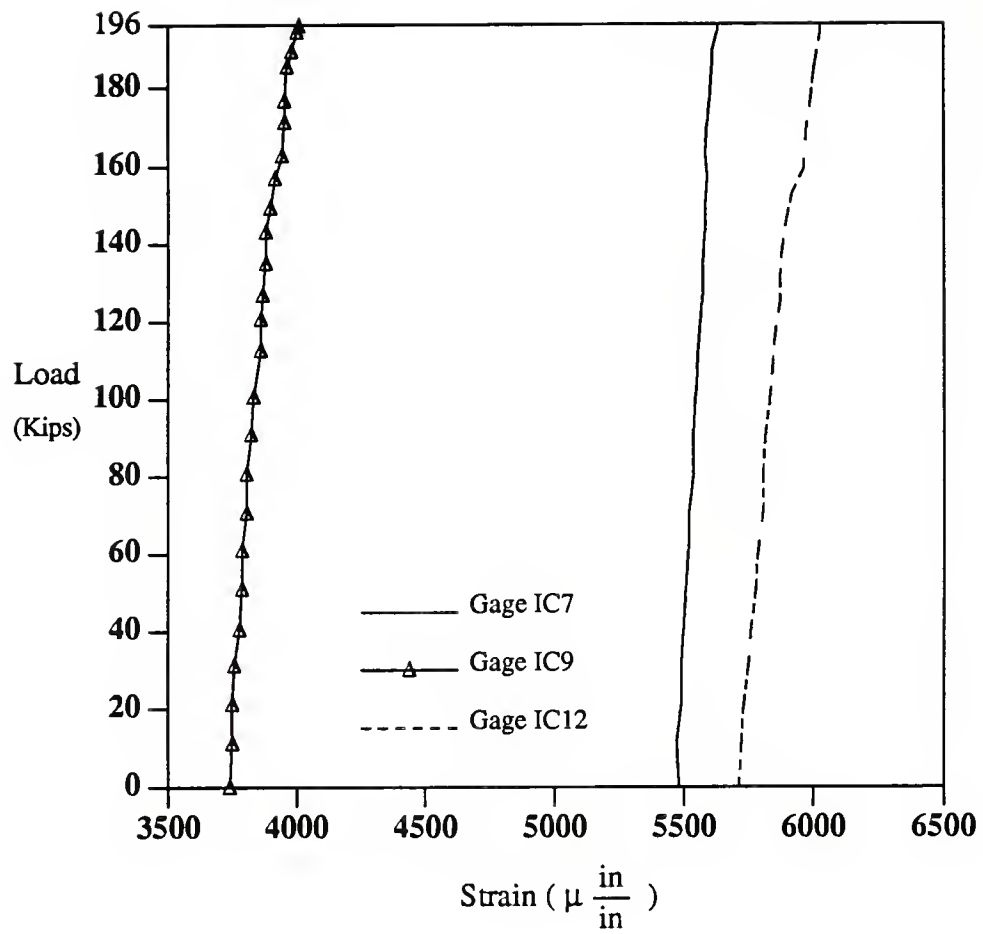


Figure 3.155 Strand Strain at 42 in from Support IC
Beam with 0% Debonding (Specimen Set 5)

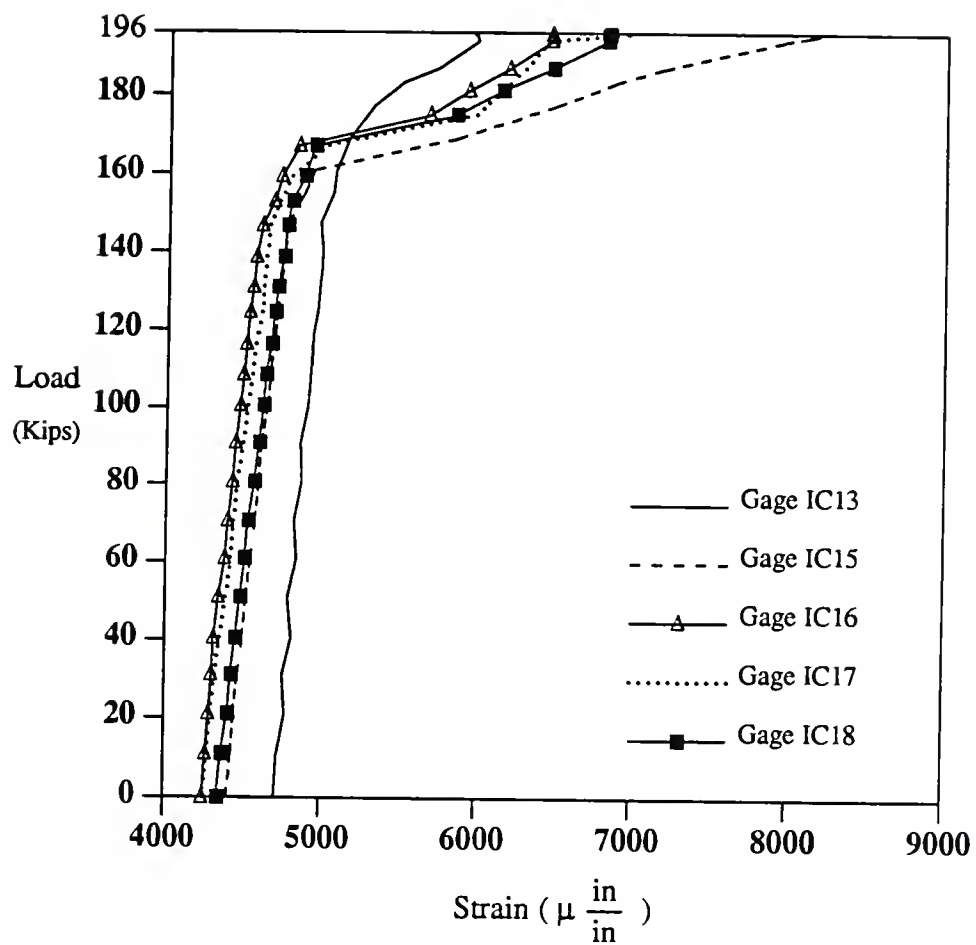


Figure 3.156 Strand Strain at 84 in. from Support IC
Beam with 0% Debonding (Specimen Set 5)

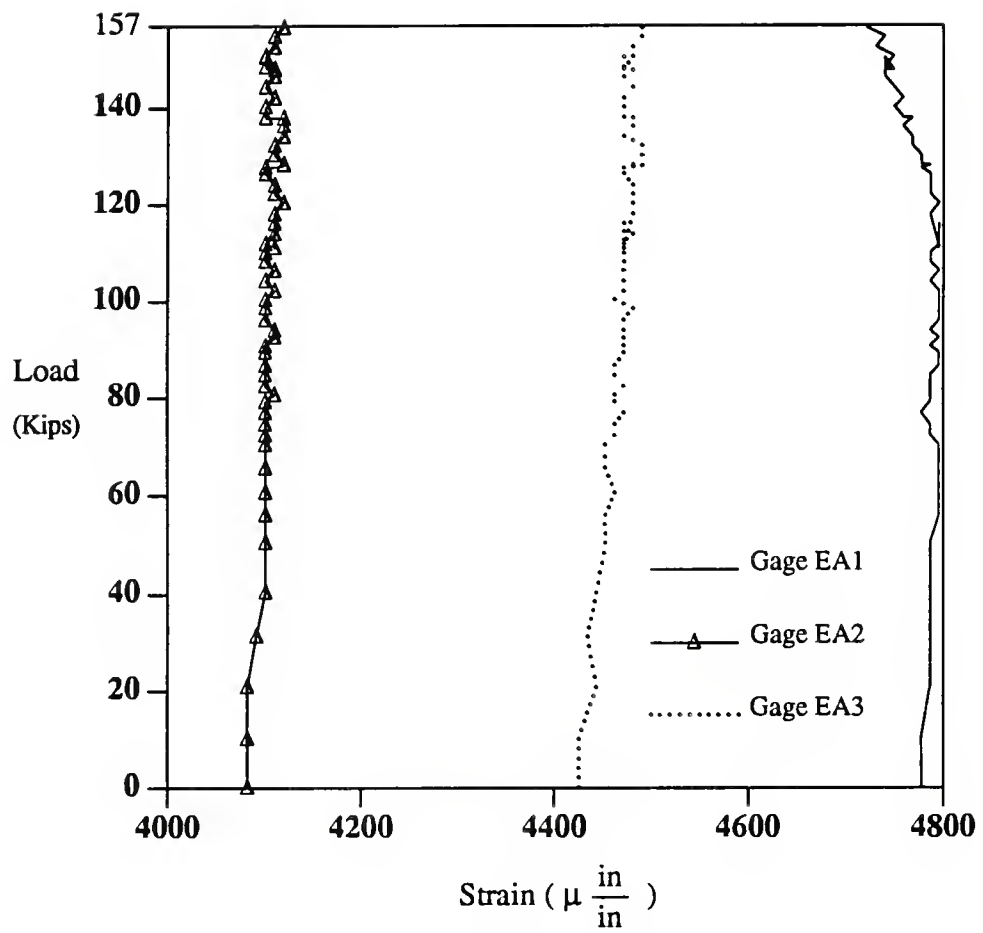


Figure 3.157 Strand Strain at 20 in from Support EA
Beam with 50% Debonding (Specimen Set 5)

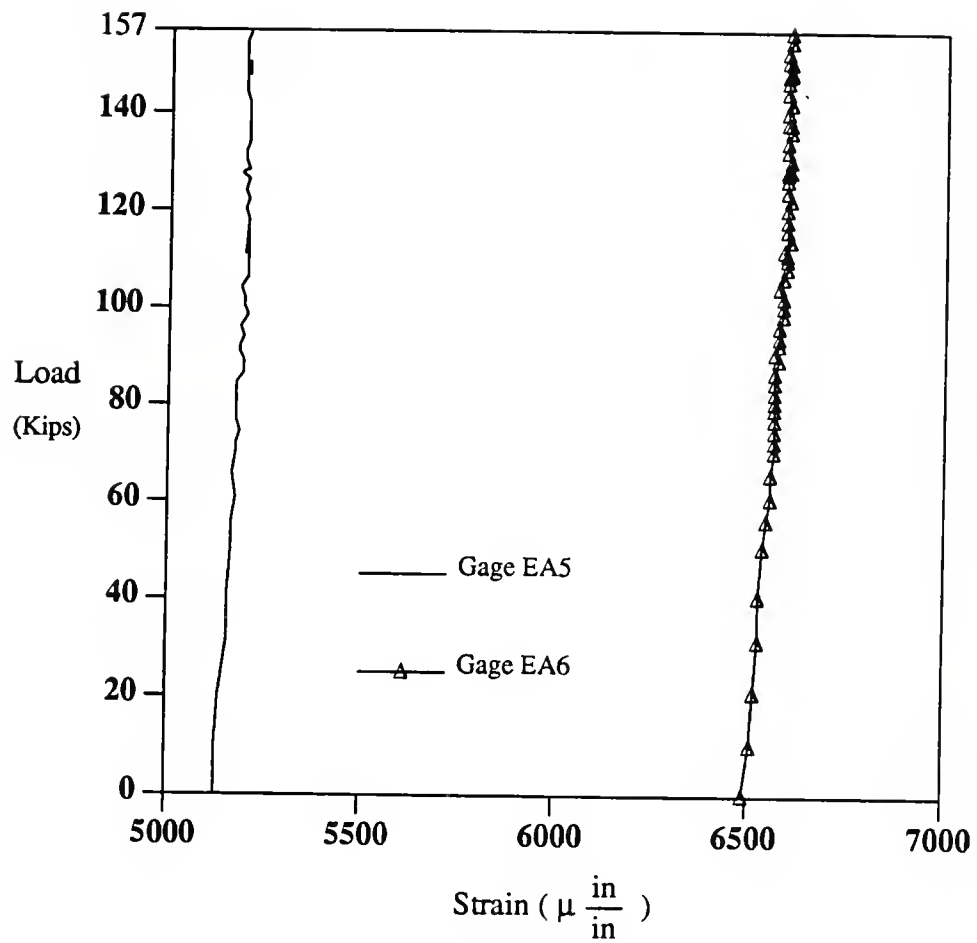


Figure 3.158 Strand Strain at 42 in. from Support EA
Beam with 50% Debonding (Specimen Set 5)

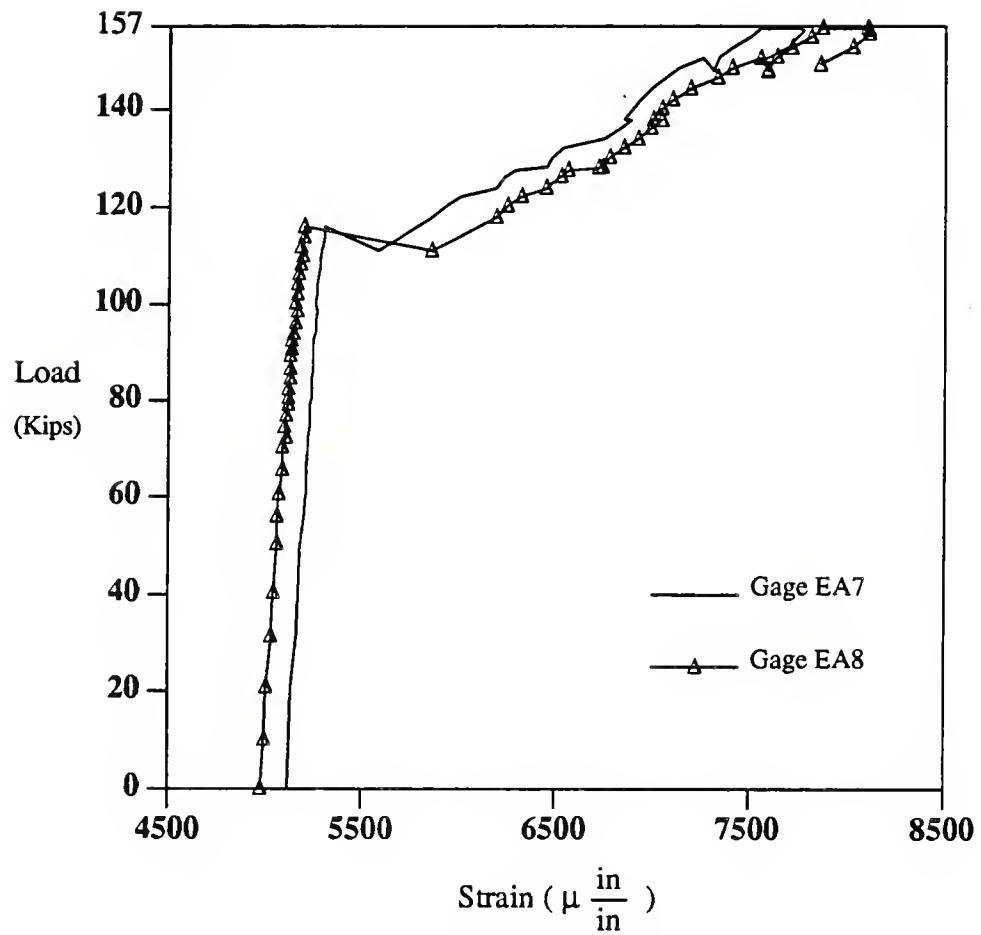


Figure 3.159 Strand Strain at Left Debonding Point
Beam with 50% Debonding (Specimen Set 5)

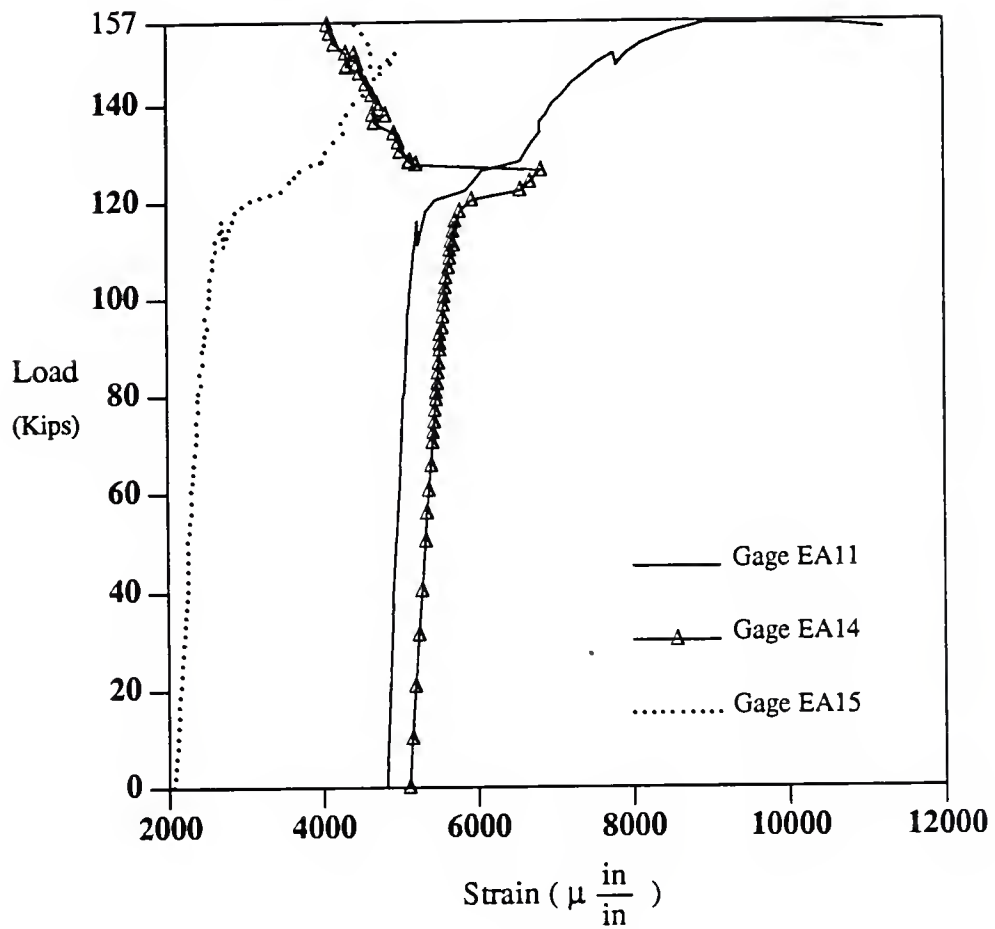


Figure 3.160 Strand Strain at the Point Load Beam with 50% Debonding (Specimen Set 5)

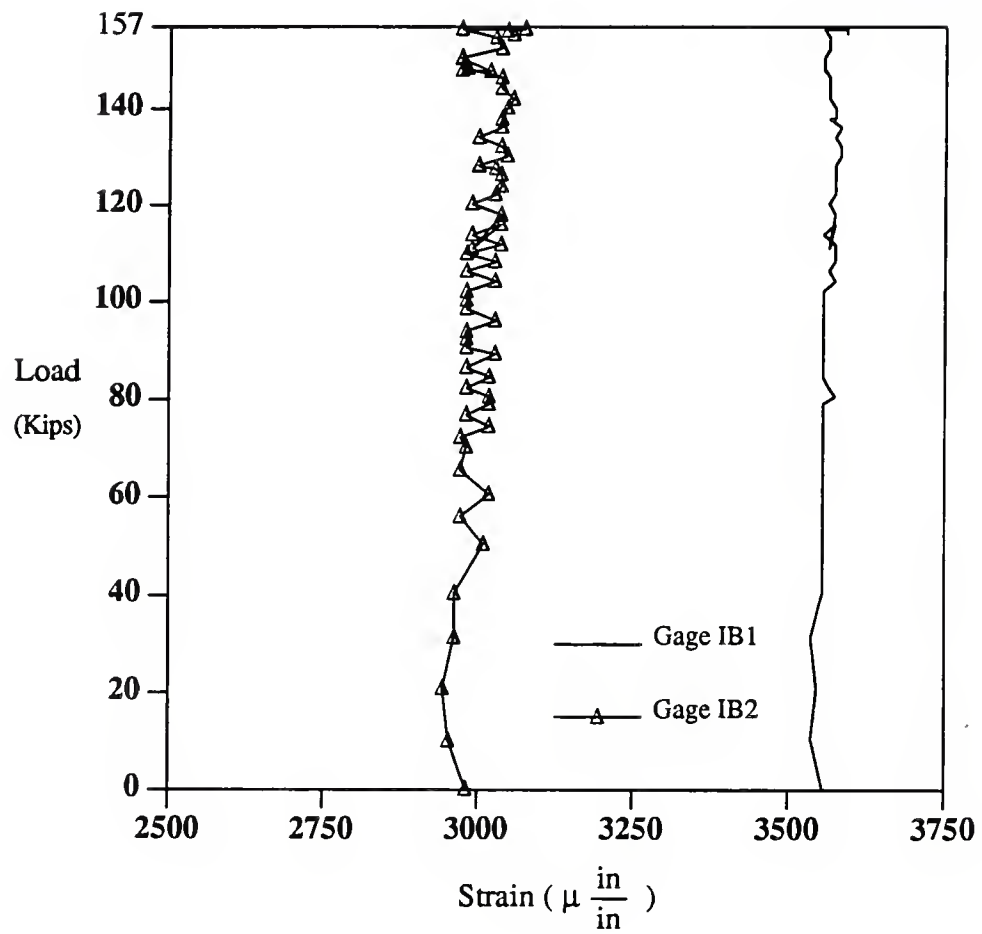


Figure 3.161 Strand Strain at 20 in. from Support IB
Beam with 50% Debonding (Specimen Set 5)

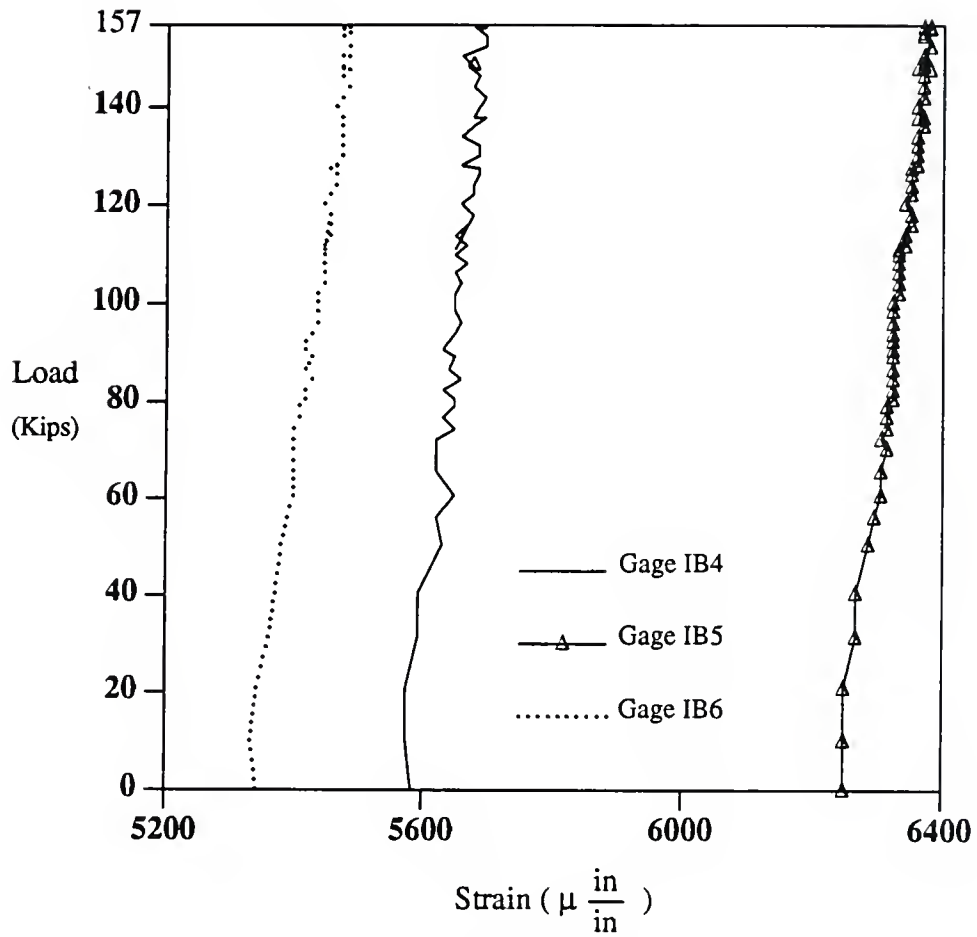


Figure 3.162 Strand Strain at 42 in. from Support IB
Beam with 50% Debonding (Specimen Set 5)

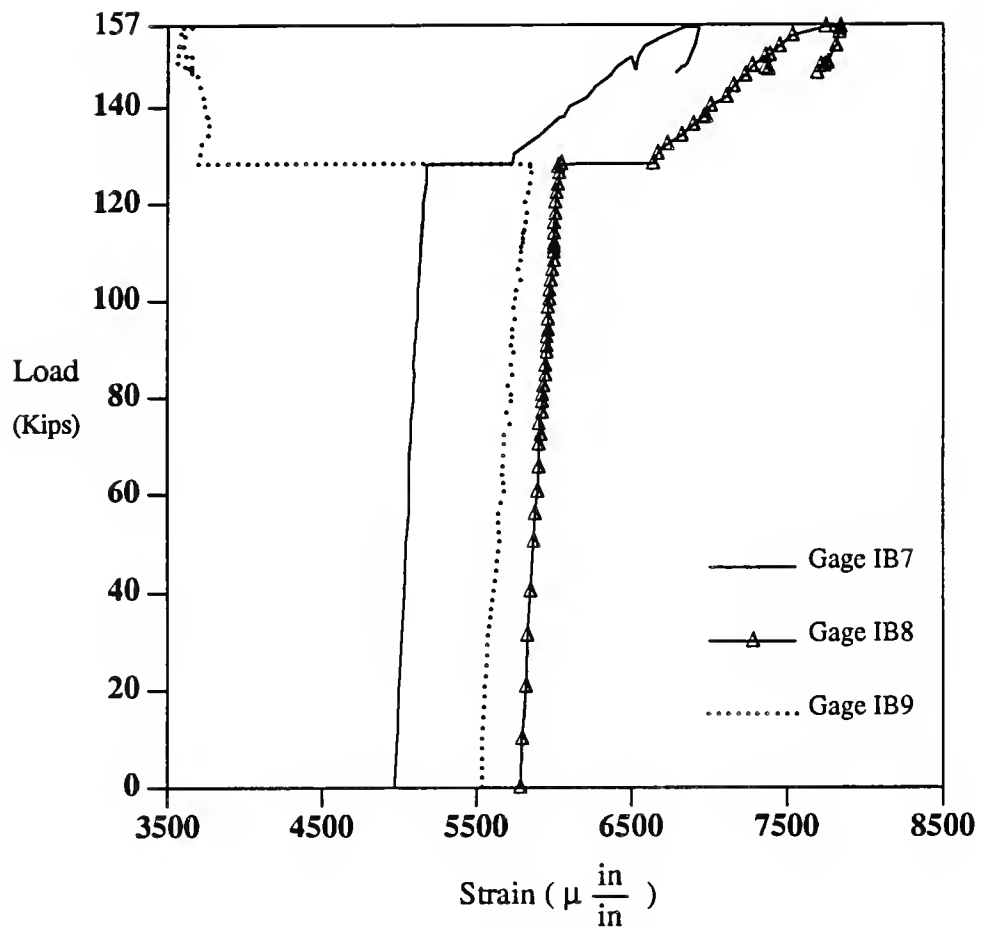


Figure 3.163 Strand Strain at Right Debonding Point
Beam with 50% Debonding (Specimen Set 5)

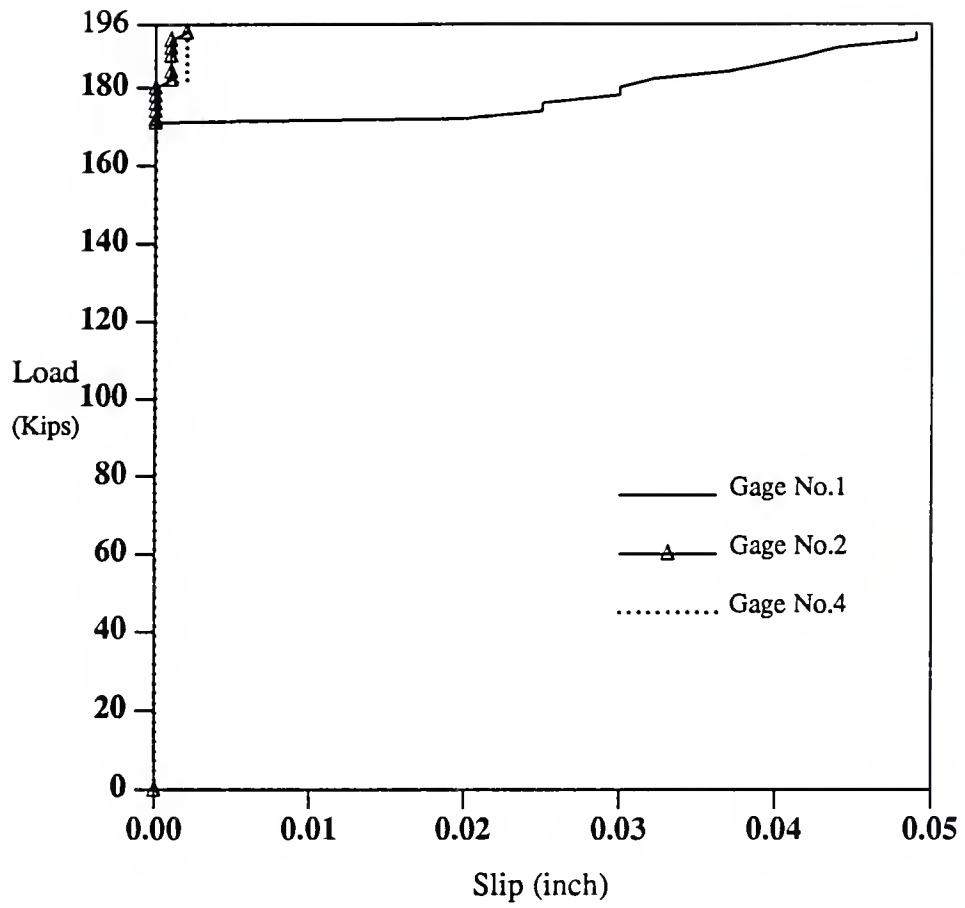


Figure 3.164 Strand Slip, Beam with 0% Debonding
Specimen Set 5

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